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SENSITIVITY ANALYSIS OF FHWA STRUCTURAL MODEL VESYS II

Vol. 1. Preparatory and Related Studies

SEP 20 1977

March 1976
Final Report

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16. Abstract This is Volume 1 of a two-volume report and includes the results of studies preparatory to a detailed sensitivity analysis for the FHWA pavement structural model VESYS IIM, comparisons of predicted and measured pavement performance and recommendations for improvements to VESYS IIM. Volume 2 reports the results of the detailed sensitivity analysis itself.

These studies preparatory to the sensitivity analysis were primarily aimed at developing realistic values for the many input variables, some of which had not been used previously in Engineering practice and for which limited practical data existed. Rational values were developed for the independent variables as well as stochastic inputs for their variations. General improvements and corrections to the mathematical models were also made to allow more accurate predictions of the responses or dependent variables. These included correction of mathematical errors in the 1) damage index model, the stochastic version of the AASHO equation and in the slope variance formulation, 2) development of means to obtain better fit from the Dirichlet Series curve fit used throughout VESYS IIM, 3) development of fatigue-temperature relationships that could be used to develop more rational fatigue cracking predictions, 4) development of new test procedures for creep compliance relationships believed to yield much more rational results and 5) improvements to the service life model.

Comparisons of VESYS IIM predicted responses to those actually measured for certain AASHO Road Test and Brampton Test Road Sections were made to test VESYS IIM capability for rational predictions.

The effort described above is discussed in detail and the results presented in this report.

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PREFACE

BY AUTHORS

This is Volume 1 of a two-volume report and includes the results of studies preparatory to a detailed sensitivity analysis for the FHWA pavement structural model VESYS IIM, comparisons of predicted and measured pavement performance and recommendations for improvements to VESYS IIM. Volume 2 reports the results of the detailed sensitivity analysis itself. These studies preparatory to the sensitivity analysis were primarily aimed at developing realistic values for the many input variables, but also included certain improvements to the computer program. Another important task was comparison of predicted performance to measured performance for four pavement sections at the Brampton Test Road and four sections at the AASHO Road Test site.

This work was accomplished by a team of engineers and other professionals including Harvey J. Treybig, Thomas W. Kennedy, R. C. G. Haas, R. Franklin Carmichael III, Harold L. Von Quintus, Robert P. Smith, Jack P. Randall, Frank Meyer, and the authors.

Support for the contract was provided by the Federal Highway Administration, Office of Research and Development, Contract No. DOT-FH-11-8258. We are grateful for the valuable technical coordination provided by Mr. William J. Kenis, FHWA Contract Manager.

J. Brent Rauhut
John C. O'Quin
W. Ronald Hudson

BY FHWA

The work under this contract was conducted to delineate those major areas where additional work would be necessary to improve the operating and predictive capabilities of the VESYS computer programs. Since the conclusion of this contract, some major changes have been made to the program by FHWA. The resulting version is called VESYS IIM(1-4). This version, along with the recently developed FHWA VESYS design users manual, has been distributed to five agencies to be used in a trial implementation design and analysis capacity and as a working tool to evaluate the potential field performance of new materials. The implementation of VESYS IIM is being conducted through Office of Development contracts and HPR studies to determine its suitability as a standard design tool for highway departments.

The findings from this work and the results obtained from new studies currently underway will be used to aid FHWA in making continuing updates and improvements to the VESYS computer program and design users manual.

T. F. McMahon, Chief, Pavement Systems Group
W. J. Kenis, Project Manager, 5C

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CHAPTER I

INTRODUCTION

Background

The FHWA Research Project "New Methodology for Flexible Pavements" has as its objective the development of a rationally based pavement design procedure, which has the capability to predict performance of the pavement over its useful life. As an initial step, a pavement analysis system was developed by Moavenzadeh and others under an FHWA contract with the Massachusetts Institute of Technology (MIT) (References 1[1], 2[2], and 3[3]). This included concepts for sophisticated design optimization procedures and a computer program called VESYS that performed a structural analysis for a three-layer pavement system on a probabilistic basis and predicted fatigue cracking, rut depth, slope variance, Present Serviceability Index (PSI), and service life with time. Subsequent improvements by MIT and the FHWA resulted in an improved version (August 1974) of the computer program called VESYS IIM. An FHWA contract initiated with the University of Utah is aimed at expanding VESYS IIM to a five-layer capability.

VESYS IIM is a comprehensive but extremely complex computer program currently requiring some 67 input values, about 27 of which are program control variables and the rest actual independent variables. It is not possible to understand clearly the characteristics of such a large simulative model except through a well defined and detailed sensitivity analysis. Accordingly, the FHWA contracted with Austin Research Engineers Inc to conduct such an analysis. The stated FHWA objective for this analysis was:

> To determine the sensitivity of the VESYS II computer program input variables on predicted pavement serviceability and to evaluate predicted serviceability in terms of realistic pavement performance.

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[1]Moavenzadeh, F., Soussou, J. E., Findakly, H. K., "Synthesis for Rational Design of Flexible Pavements", Part I, School of Engineering, Massachusetts Institute of Technology, Cambridge, Mass., 02139, January 1973.

[2]Soussou, J. E., F. Moavenzadeh and H. K. Findakly, "Synthesis for Rational Design of Flexible Pavements, Part II", FHWA Contract No. FH-11-776, January 1973.

[3]Moavenzadeh, F., Soussou, J. E., Findakly, H. K., Brademeyer, B., "Synthesis for Rational Design of Flexible Pavements", Part III, "Operating Instructions and Program Documentation", February 1974.

In addition to the sensitivity analysis, FHWA wished to learn if the pavement performance predicted by VESYS IIM realistically reflects performance measurements on real pavements and to obtain recommendations for improvements to VESYS IIM.

Work Preparatory to the Sensitivity Analysis

The development of the sophisticated mathematical idealization called VESYS IIM for flexible pavement structures has led to definition of independent variables not previously used in engineering practice and for which limited practical data exist. Also, limited stochastic data exist for other independent variables, even those in more common use. If the sensitivity analysis is to be meaningful, all input variables must be varied over realistic and consistent ranges. Also, the various mathematical models comprising VESYS IIM must function properly. The major portion of the research effort was aimed at defining realistic input values for the independent variables and improving or correcting the mathematical models to predict observed performance responses more accurately (the modified version of VESYS IIM resulting was dated June 1975). This effort preparatory to conducting the sensitivity analysis is reported in this volume (Volume 1) of this project report. The results from (1) comparisons of calculated performance responses to field measurements from the AASHO Road Test and the Brampton Test Road, (2) identification of limitations in the VESYS IIM system and (3) recommendations for research needed to improve it also appear in Volume 1.

The Sensitivity Analysis

There is no established procedure for conducting a sensitivity analysis for systems having up to 30 independent variables. A full factorial consideration would entail in the order of 3^{30} VESYS IIM solutions for three levels of each independent variables or 2^{30} for two levels, either number being impossibly large. It was necessary to (1) screen out relatively insignificant independent variables in stages, and (2) by various rational methods separate the analysis into two factorials, (3) use fractional factorial techniques and (4) use other carefully constructed innovations to reduce the task to a manageable size.

The sensitivity analysis was completed with a very minimum of lost information despite the staggering sizes of the full factorials represented. Separate analyses were conducted for (1) cracking damage, (2) rut depth, (3) slope variance, (4) Present Serviceability Index (PSI) and (5) Service Life. The procedures employed, details of the specific analyses and the results of the sensitivity analysis are described separately in Volume 2.

CHAPTER II

BRIEF DESCRIPTION OF VESYS IIM

The VESYS II computer program and its capabilities have been described in some detail in references 4[1] and 5[2]. However, VESYS IIM has been revised considerably from the original VESYS II program, and some discussion on how it operates is essential to the understanding of this report. A diagram of the VESYS IIM pavement analysis and design system is shown in Figure 1. As can be seen, the input variables arrayed vertically at the left side of the diagram are partially stochastic and partially deterministic. Material properties inputs are fit to a curve using a Dirichlet series formulation so that the material characteristics are expressed in equation form for ready mathematical manipulation. The middle portion of the diagram shows a stationary load subprogram and a repetitive load subprogram. The stationary load subprogram is basically elastic layer theory applied to calculate stresses, strains, and displacement at the top and bottom of the asphaltic concrete layer, at the top and bottom of the base layer, and at the top of the subgrade layer. When a unit load is applied to a specified loaded area at the surface, one solution is then obtained for each load duration using the set of stiffnesses obtained for the three layers from their creep-compliance vectors. Calculated displacements at the top of the surface are then expressed in a Dirichlet series form as a function of load duration or time. Radial strains at the bottom of the surface layer are similarly expressed as a function of time. The result at this stage is a set of equations representing total displacements at the surface and radial strains at the bottom of the surface layer, each as a function of loading time to reflect analytically the viscoelastic nature of the pavement materials.

A second series of similar static solutions are then run using values of layer elastic moduli derived by revising creep compliance relationships to omit permanent strains (basis is repetitive load testing to determine permanent strain as a function of number of loads applied). In explanation, the creep compliance values input to the computer program are based on total strains which are a combination of elastic and permanent strains.

[1]Kenis, W. J. and T. F. McMahon, "Pavement Analysis System VESYS II," Paper prepared for AASHO Design Committee Meeting, October 1972.

[2]Kenis, W. J. and Dr. T. F. McMahon, "A Flexible Pavement Analysis Subsystem," Paper presented at 53rd Annual Meeting of Highway Research Board, January 1974.

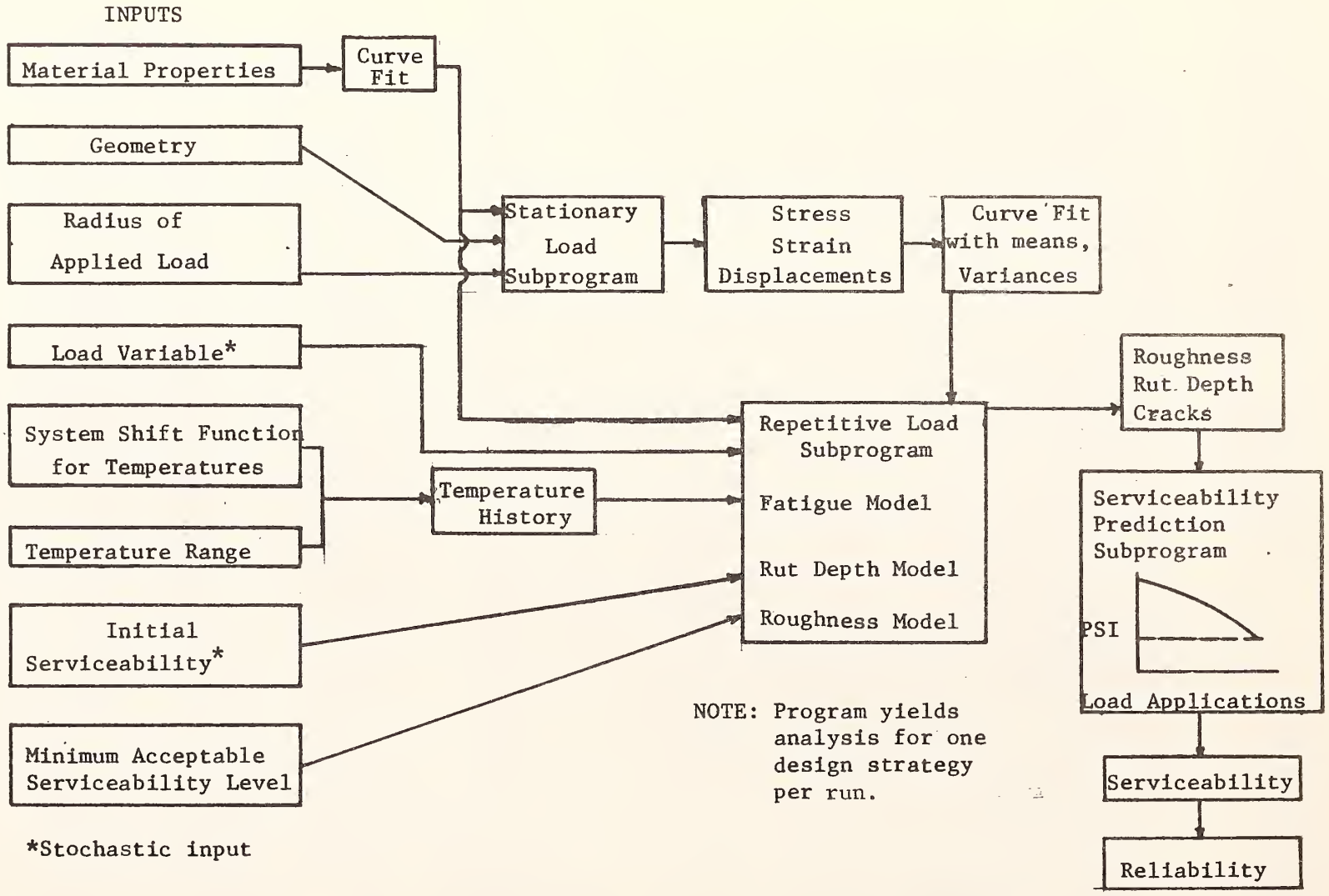

Figure 1. VESYS II M pavement analysis and design system (Ref 32).

The revised creep compliance values are based on elastic strains alone. We now have a second set of equations describing elastic or recoverable displacements. The differences of displacement at the surface between the two series of calculations may be taken to be the permanent strain or rutting at different load durations for one load cycle.

As permanent strain changes with number of load cycles applied, it is reasonable to calculate permanent deformation for various numbers of load applications so that displacements at the surface may be expressed as a function of both load duration and number of cycles.

Having the actual load duration and a time-temperature shift relation for the asphaltic concrete surface layer, a revised load duration that considers actual load duration and temperature effects on the creep compliance can be obtained. Application of these revised load durations to the sets of equations previously discussed also imposes the same revised load durations on the creep-compliance vectors for the base and subgrade, but the effect is minor as creep is small in these materials.

Sufficient information has been accumulated in the storage arrays at this point in the program from which to calculate the permanent deformations that occur at the surface and under the center of load. Separate calculations are made for each interval of time (usually one month) with its separate value of temperature specified and the corresponding number of loads or traffic during that time interval.

The analysis moves forward in time, using consistent revised load duration values for the temperature data, the number of cycles that have already occurred at the beginning of the time interval and those that will occur during the time interval to predict the accumulation of permanent displacements (rutting) at the surface since construction. Printout of these values may be made at points in time specified by the computer program input TRANDØM.

The prediction of slope variance required for the AASHO equation for present serviceability index is obtained by statistical means on the basis of the stochastic input variables according to the following equation:

$$E[SV] = \frac{2B}{C^2} \cdot V[GNORMAL]^2 \, (E[RD]^2 + Var[RD]) \qquad (1)$$

where:

E[SV] ≃ Expected slope variance (radians x 10^6)

= mean slope variance

B = Materials variable called CØRLCØEF for which the value 1 may be used

C = Material variable called CØRLEXP

V[GNORMAL] = Coefficient of variation for the array of curve-fitted Dirichlet coefficients for normal time-dependent deflections at the surface as a function of load duration.

E[RD] = Mean rut depth

Var[RD] = Variance of rut depth

The variance of slope variance is predicted as:

$$\text{Var[SV]} \simeq \frac{16B^2}{C^4} \cdot \text{V[GNØRMAL]}^4 \cdot \text{E[RD]}^2 \cdot \text{Var[RD]} \qquad (2)$$

The very complex and laborious derivations of these equations are based on probability theory and appear in Reference 2:

Fatigue calculations for estimating damage due to cracking are considerably less complex than those for rutting. Damage may be accumulated according to Miner's Hypothesis as follows.

$$D_j = \sum_{i=1}^{j} \frac{n_i}{N_i} = \sum_{i=1}^{j} \frac{\varepsilon_i^{K_{2i}}}{K_{1i}} \lambda_i \, (t_i - t_{i-1}) \qquad (3)$$

where:

n_i = Number of load cycles during the ith time interval

$= \lambda_i \, (t_i - t_{i-1})$

N_i = Number of cycles to failure under temperature and strain conditions of ith time interval

$= K_{1i} \left(\frac{1}{\varepsilon_i}\right)^{K_{2i}}$

D_j = Mean damage index at time t_j

ε_i = Mean strain during the ith time interval,

λ_i = Traffic rate during the ith time interval,

t_i = The ith element of the TRANDØM array, $t_o = 0$,

K_{1_i} = Fatigue relation coefficient for ith time interval in terms of temperature

K_{2_i} = Fatigue relation exponent for ith time interval in terms of temperature.

The fraction of damage or cracking induced during each time interval is accumulated. Cracking failure is considered to occur when $D_j = 1$.

The coefficient K_1 and the exponent K_2 that define fatigue life in relation to initial strain at the bottom of the surface layer are temperature dependent and different values are input for each temperature value used.

As the fatigue materials characteristics are treated stochastically, the actual formulation used in VESYS IIM is:

$$E[D_j] = \sum_{i=1}^{j} n_i \, E\left[\frac{1}{N_i}\right] = \sum_{i=1}^{j} \lambda_i \, (t_i - t_{i-1}) \left\{ \frac{\varepsilon_i^{K_{2i}}}{K_{1_i}} \left[+ \frac{\text{Var } K_{1_i}}{(K_{1_i})^2} - \frac{\text{Cov}\left[K_{1_i}, K_{2_i}\right] \cdot \ln \varepsilon_i}{K_{1_i}} + \frac{\text{Var}\left[K_{2_i}\right] (\ln \varepsilon_i)^2}{2} \right] + \frac{K_{2_i}(K_{2_i}-1) \, \varepsilon_i^{(K_{2_i}-2)} \text{Var}[\varepsilon_i]}{2K_{1_i}} \right\} \quad (4)$$

Where $\text{Cov}[K_{1_i}, K_{2_i}]$ = covariance of K_{1_i} and K_{2_i}.

This is the expected value of the Taylor series expansion for D_j about the means of K_{1i}, K_{2i} and ε_i. In this expression, the ε_i are assumed to be independent of the K_{1i} and the K_{2i}. The variance of D_j (Var $[D_j]$) can be obtained in a similar fashion, taking the second moment about the mean of the Taylor series expansion.

It is assumed that D_j is the value of a normally distributed random variable with mean $E[D_j]$ and variance $Var[D_j]$ as indicated in Figure 2. The cumulative distribution function (c.d.f.) for this variable at a point D_o is:

$$F(D_o) \int_{-\infty}^{D_o} f(t)d_t = (2\pi \text{ Var}[D_j])^{-\frac{1}{2}} \int_{-\infty}^{D_o} e^{-\left[\frac{(t-E[D_j])^2}{2 \text{ Var}[D_j]}\right]} dt \qquad (5)$$

where

$f(t)$ is the probability density function (p.d.f.) of damage at the point t (assumed normal), and

$F(D_o)$ is the c.d.f. of damage at the point D_o

$F(D_o)$ can be interpreted as the probability that the damage index of an arbitrary square yard of the pavement surface is less than or equal to D_o. Hence, under Miner's Hypothesis, F(1) is the probability that the specimen has not failed (i.e. does not exhibit cracking distress). Therefore:

$$C = 1000\,[1-F(1)] \qquad (6)$$

where:

C is the expected area cracked in square yards per 1000 square yards.

The present serviceability index (PSI) is calculated from a stochastic version of the classical AASHO equation that yields the expected value of PSI as follows:

$$E[PSI] = 5.03 - 1.91 \quad E[\log_{10}(1 + SV)] - .01E[\sqrt{C + P}]$$

$$- 1.38\ E[RD^2] \qquad (7)$$

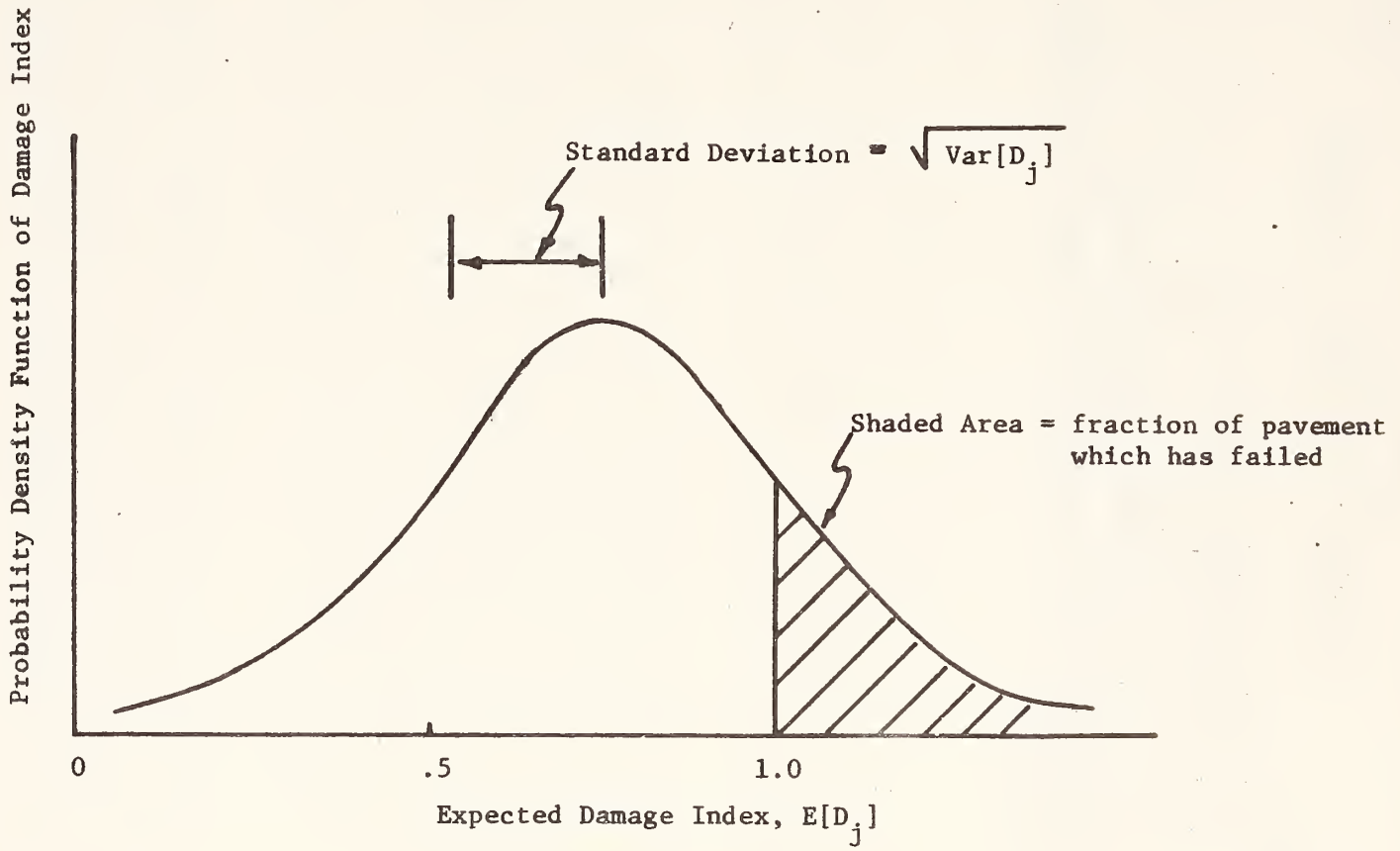

Figure 2. Normal probability density function used to predict area cracked

$$E[PSI] \simeq 5.03 - 1.91\left[\log_{10}(1 + E[SV]) - \frac{Var[SV] \cdot \log_{10}e}{2(1 + E[SV])^2}\right]$$
$$-.01(E[C])^{\frac{1}{2}} - 1.38\,(E[RD]^2 + Var[RD]) \qquad (8)$$

where:

SV = slope variance, radians x 10^6

RD = rut depth, inches

C = area of cracking, sq yds/1000 sq yds

The area of patching and the variation of cracking are assumed to be zero in the program.

CHAPTER III

INPUT VARIABLES FOR VESYS IIM

VESYS IIM as currently configured normally requires 47 input variables, 12 of which include multiple values in arrays. For simplicity, these input variables may be classed into four categories having rational engineering significance when considered separately. These classifications may include pavement thickness dimensions, traffic and wheel load data, materials characterizations and control variables.

Pavement Thickness Dimensions

As VESYS IIM uses a three-layer elastic layer system, it is only necessary to describe the thicknesses of the surface layer of asphaltic concrete and the base layer as the thickness of the subgrade will be infinite because of the underlying assumption of a semi-infinite half-space.

When a pavement system with more than three layers is to be evaluated, it is necessary to combine similar layers to reduce to the three allowed for analysis. For example, if an asphalt-treated base (ATB) and a granular subbase both exist, it would seem logical to combine the asphalt-treated base with the asphaltic surface layer. While the characterization of such a combination must be based on judgment, it appears reasonable to use an interpolated creep compliance curve between those for the two materials, but to use the fatigue life data for the ATB because the tensile stresses leading to fatigue failure and propagation of cracks upward would primarily occur in this lower portion of the combined layer. This assumes an efficient tack coat that would create sufficient bond that the two layers would work more or less as a single layer.

In the case of a base and subbase of granular materials, it would be reasonable to combine these two layers into one layer with an interpolation for a creep compliance curve and values of permanent deformation coefficients. If the upper subgrade layer is reasonably thin (say one to two feet), it will be necessary to combine subgrade layers as well, but such a combination would not affect any pavement dimension input.

Traffic and Wheel Load Data

Traffic is defined in terms of axles per day as an array corresponding to another array of points in time after construction for

which the various responses are to be calculated. This allows consideration of increased or decreased traffic conditions at various times throughout the analysis period.

Because of the viscoelastic characterization with the VESYS IIM analysis, the duration of the load must be defined. The value input to VESYS IIM is the mean duration of the repeated loadings. The estimation of this value can not presently be based on very clear guidelines. It is quite clear that the variation of the duration of loading involves the period for which the vehicle's weight has a significant effect on the layer under consideration, and that this varies with each layer and in fact at different depths within each layer. It has been suggested that the duration be taken as the time required for the vehicle to traverse three times the diameter of the loaded area at the velocity of its travel. It appears to the authors, however, that a lesser period should be adopted more consistent with the surface layer alone as the primary permanent deformation effects appear in that layer (including asphalt-treated base if existing in the pavement structure). Fortunately, the responses for VESYS IIM are fairly insensitive to this variable in the practical range.

Variability of the duration of loading is considered through input of its variance called VCDUR. Verified equations for calculating VCDUR do not exist. This variance must be calculated in terms of variations in the radius itself (function of the wheel load and tire pressure) and variations in the velocity of truck traffic. Analysis is required using judgement, existing data, and innovations. As for DURATION, VESYS IIM solutions are insensitive to VCDUR in the practical range.

The wheel load itself is defined by the input of the value of RADIUS in inches, a variable called AMPLITUD in psi and the variance of AMPLITUD. While AMPLITUD is treated internal to the computer program as tire pressure, the fact that the radius is fixed requires it to also represent the range of wheel weight as well. The value of AMPLITUD to be input is one half of the mean axle weight in pounds divided by the loaded area (πr^2). As the radius is fixed in the program and the only variable is wheel weight, the variance of AMPLITUD may be readily determined in terms of variance in the wheel weight distribution.

Materials Characterization

All three layers are characterized individually in terms of their viscoelastic stiffness and their permanent deformation characteristics. The surface layer is characterized additionally in terms of its fatigue life characteristics. New methods of testing are required to arrive at these input values. Such methods are generally familiar only so researchers and not to practicing engineers.

Viscoelastic Stiffness Characterization

Elastic layer theory requires that each layer be identified in terms of its thickness, its modulus of elasticity and Poisson's ratio. As the solutions are only mildly affected by Poisson's ratio, representative constant values are applied in the program. The modulus of elasticity then becomes the primary parameter for each layer, but for viscoelastic materials, it is variable depending on duration of loading. Creep compliance, the inverse of modulus of elasticity, is commonly used by researchers to define the variation of the modulus of elasticity with load duration. These values are obtained by applying a static constant load to a test specimen and measuring the total strain at different points in time. These data are input for the three layers as arrays of values correlated to a single array of time values. As stiffness or modulus of elasticity for the surface asphaltic concrete layer is a function also of temperature, a time-temperature shift function BETA is also used (see Appendix E for description).

The computer program converts the creep compliance-time arrays into Dirichlet series equations for ease in mathematical manipulation. The effects of temperature variation are accounted for by simply shifting via the available data to a different time duration on the 70°F curve that will give the same value of creep compliance as the real time duration on a curve representing the real temperature.

It is important to note at this point that an unload curve measured during a creep compliance test is not an input to this program. The difference between the so-called load and unload values of creep compliance multiplied by the test stress indicates the permanent strain occurring during the test. This is sometimes used as a basis for predicting rutting. However, this is simply for a single load and will grossly overpredict rutting as the amount of permanent deformation for a single cycle is grossly reduced as repeated loading continues.

Permanent Deformation Coefficients

Two permanent deformation coefficients (really one multiplier and one exponent) are developed through materials testing for each layer. They are called ALPHA and GNU and simply represent convenient mathematical parameters for fitting the relations of permanent strains to cycles of load to a curve on a log-log plot. As these relations are completely new to the engineering profession, they are derived below for clear understanding.

The characterization of a material's permanent deformation potential under repeated loads is central to its participation in the

rutting appearing at the surface of the pavement structure. A specimen loaded in the laboratory will generally exhibit resilient or recoverable strain and nonrecoverable strain during a load cycle. The resilient strains usually stabilize to a constant value after 200 to 1000 load cycles, dependent on the material itself. The permanent strain per cycle is relatively large at the beginning of the test and continually decreases as the repeated loads continue unless approaching fatigue failure. The accumulated permanent strains may be plotted versus numbers of load cycles as in Figure 3 which illustrates a typical case. Note that about 50% of the permanent strain occurring in 700,000 cycles had already occurred at 100,000 cycles.

The same data may be plotted on a log-log plot as in Figure 4. Some means is needed to relate permanent strain for a single load cycle to the number of previous load cycles so that the permanent strain during any load cycle may be predicted. It is also important to use some method that will provide the greatest accuracy in the region of interest, usually well past the number of cycles applied in testing as design life traffic will usually exceed practical limits on repetitive loads during laboratory testing. The region of highest interest is then that near the end of the test.

The method selected to represent permanent deformation characteristics of the materials for VESYS IIM involves in effect a linear curve-fit on the log-log plot passing through the data late in the test as shown in Figure 4. This line may be defined by its intercept, I at load cycle 1 and its slope S. Applying the equation for a line:

$$\log \varepsilon_a = S \log N + \log I \tag{9}$$

$$\log \varepsilon_a = \log N^S + \log I = \log IN^S \tag{10}$$

$$\varepsilon_a = IN^S$$

The desired permanent strain due to the N^{th} loading is then

$$\varepsilon_p(N) = \varepsilon_a(N) - \varepsilon_a(N-1) \tag{11}$$

Converting the right side of Equation (11) from a discrete variable to a continuous variable and using Equation (10):

$$\varepsilon_p(N) = \frac{d\ \varepsilon_a(N)}{dN} = \frac{d\ (IN^S)}{dN} \tag{12}$$

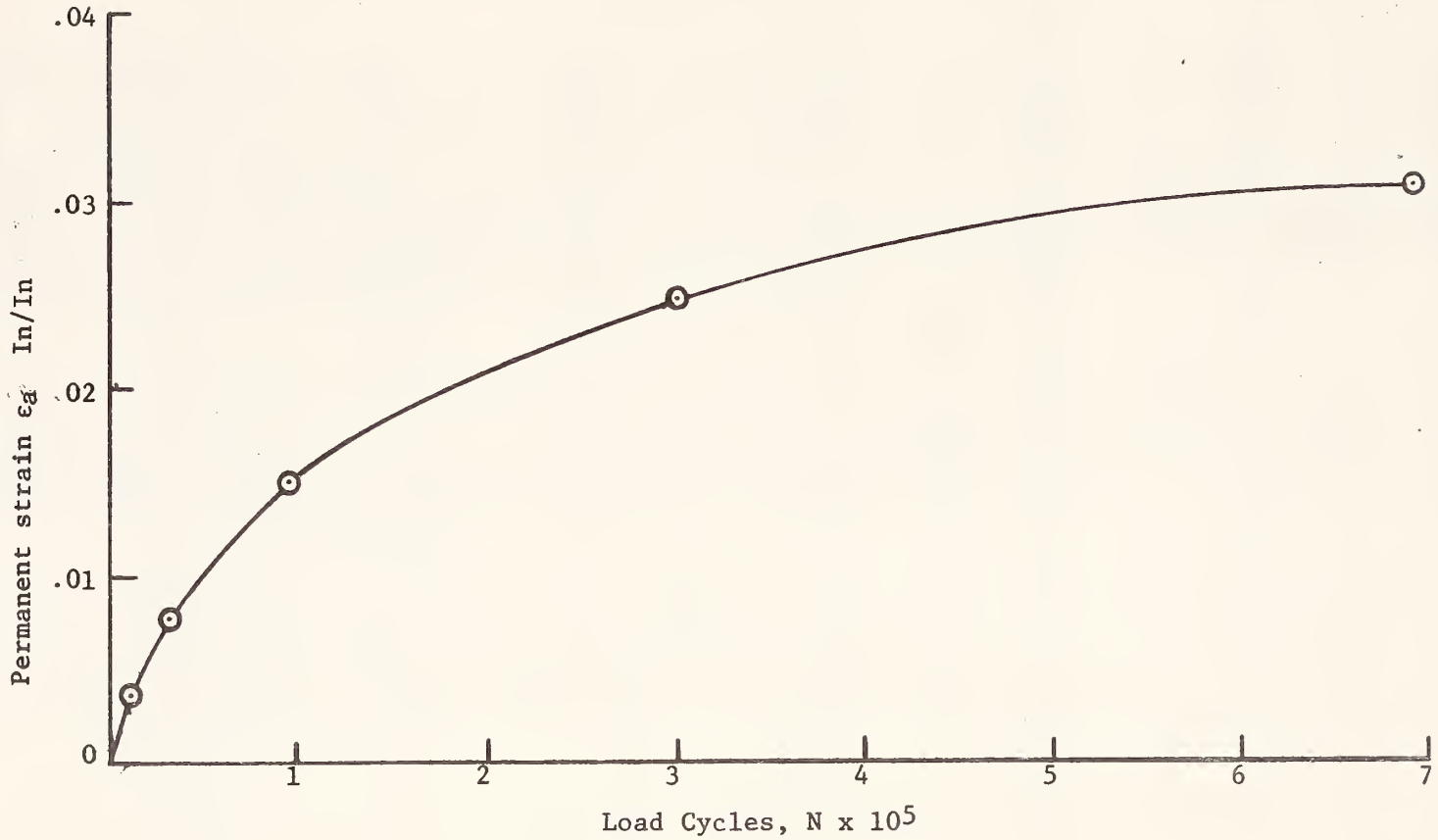

Figure 3. Typical test results for repetitive load tests on an asphaltic concrete.

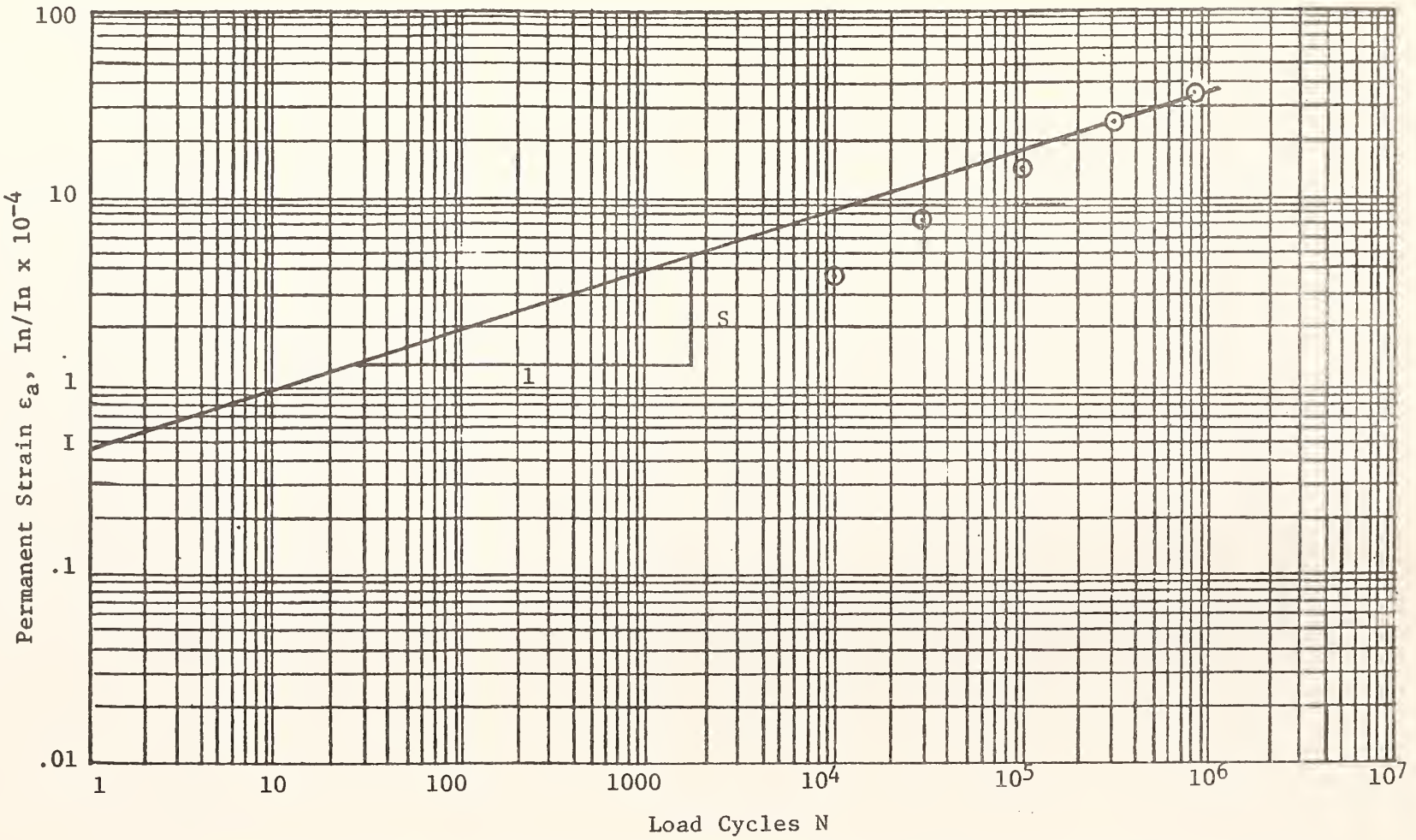

Figure 4. Typical test results for repetitive load tests on an asphaltic concrete.

$$\varepsilon_p(N) = ISN^{S-1} \tag{13}$$

As the resilient or elastic strain ε_r is essentially a constant after relatively few cycles and is large compared to the permanent strain (i.e., resilient strain is approximately equal to the total strain ε_t), the fraction of the total strain F(N) that is permanent may be considered to be:

$$F(N) = \frac{\varepsilon_p(N)}{\varepsilon_t(N)} = \frac{\varepsilon_p(n)}{\varepsilon_r} = \frac{ISN^{S-1}}{\varepsilon_r} \tag{14}$$

For convenience, arbitrary definitions may be made for mathematical simplification:

$\mu = IS/\varepsilon_r$ (called GNU in VESYS IIM)

$\alpha = 1-S$ (called ALPHA in VESYS IIM)

Equation (14) becomes:

$$F(N) = \mu N^{-\alpha} \tag{15}$$

Applying this method to the test results plotted:

$I = .00047$

$S = .31$

$\varepsilon_r = .0000776$

$$\mu = \frac{IS}{\varepsilon_r} = \frac{(.00047)(.31)}{.0000776} = 1.88$$

$\alpha = 0.69$

As F(N) is the fraction of permanent strain during cycle N, the permanent strain in a compression specimen during cycle N is F(N) multiplied by ε_r. The increment of permanent strain $\Delta\varepsilon_a$ may also be calculated during any interval of loading N_1 to N_2 by integrating $\varepsilon_r F(N)$ as follows:

$$\Delta\varepsilon_a = \int_{N_1}^{N_2} \varepsilon_r F(N)dN = \varepsilon_r \mu \int_{N_1}^{N_2} N^{-\alpha}dN \tag{16}$$

$$\Delta\varepsilon_a = \varepsilon_r \mu \left[\frac{N^{(1-\alpha)}}{1-\alpha}\right]_{N_1}^{N_2} = \frac{\varepsilon_r \mu}{1-\alpha}\left[N_2^{1-\alpha} - N_1^{1-\alpha}\right] \qquad (17)$$

Using Equation (17) for N_2 = 700,000 and N_1 = 300,000 for the data plotted on Figure 4:

$$\Delta\varepsilon_a = \left[\frac{(.0000776)(1.88)}{1 \quad .69}\right]\left[(700000)^{1-.69} - (300,000)\right]^{1-.69}$$

$$= .0071 \text{ in/in.}$$

Comparison to the increment of strain indicated on Figure 4 between the same points on the line indicates good agreement.

The total permanent shortening in a specimen of Height H would be $H \cdot \Delta\varepsilon_a$ during the increment of repetitive loading N_1 to N_2.

The importance of the fraction of permanent strain F(N) is that it is considered to be equal to the fraction of the strain during a creep compliance test not recoverable when the load is removed as shown in Figure 5. As the plot of Figure 5 would be a plot of creep compliance D(N) if the values of strain were divided by the essentially constant stress , the fraction F(N) may similarly be related to creep compliance as follows:

$$F(N) \simeq \frac{\varepsilon_p(N)}{\varepsilon_t(N)} = \frac{(\varepsilon_t(N)/\sigma) - \varepsilon_r/\sigma}{\varepsilon_t(N)/\sigma}$$

$$F(N) \simeq \frac{D_{Load}(N) - D_{Unload}(N)}{D_{Load}(N)} = \mu N^{-\alpha} \qquad (18)$$

$$\text{or} \quad D_{Unload}(N) = D_{Load}(N)\left[1 - \mu N^{-\alpha}\right] \qquad (19)$$

Using the elastic layer model, solutions can be obtained for a unit static load for the case of input values of creep compliance D_{Load}, for each layer and for the values of D_{Unload} calculated from Equation (19) for each layer. The difference in the calculated displacements at the surface under the load is considered to be the permanent displacement of the structural system.

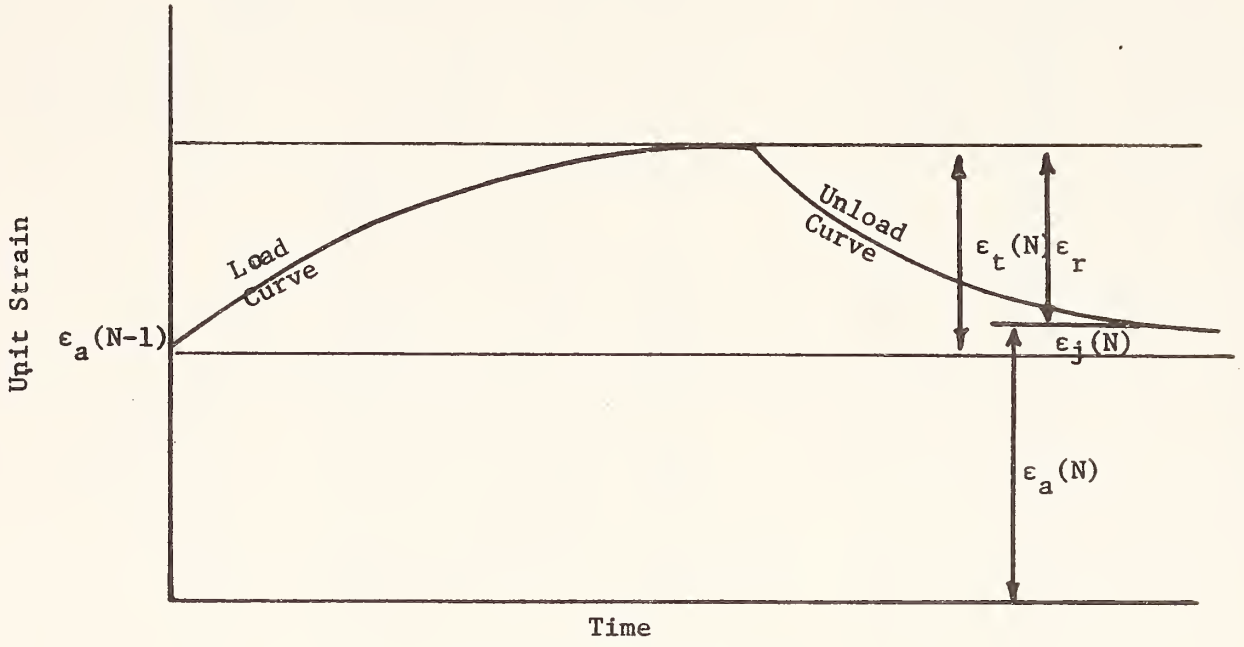

Figure 5. Plot of strain

These system permanent displacements may then be in effect plotted as in Figure 4 as a function of number of load cycles and values of "System ALPHA and GNU" obtained to predict system permanent displacements or rutting through integration much as illustrated previously for Equation (17).

Both μ and α are considered by VESYS IIM to be constant for a layer of material, but in reality they are very stress-dependent. This means that μ and α vary with depth in the layer and also laterally from the center of load to some extent. As a rational approximation, average deviator stresses for testing may be obtained from elastic layer solutions using the correct loading and approximate values of elastic moduli consistent with the materials and the load duration.

For heavily traveled highways that may have millions of wheel loads in a wheel path, the parameter ALPHA will be underestimated and rutting will be overpredicted (as the slope of the line will be too large) by test results for a lesser number of load cycles. GNU will also be affected by the lesser number of load cycles during testing and will also be underpredicted. Some improvement is desirable in development of ALPHA and GNU to represent higher values of N.

Fatigue Characteristics

The potential for fatigue cracking of the surface layer of asphaltic concrete (only layer characterized for fatigue) is the classical one of a straight line on a log-log plot of initial strain for the test versus the number of cycles to failure. Such a relation allows the establishment of the number of loads producing a specific level of strain that induce a condition defined as failure. In the classical beam test, failure shortly follows the appearance of a crack at the bottom of the beam, but in reality for asphaltic pavement layers a very considerable number of cycles will be required to propagate the cracks to the surface in sufficient number to greatly affect ride quality and be defined as failure. In either case, the input variables are the same but their values would differ greatly.

$$N_f = \text{STRNCOEF} \left(\frac{1}{\varepsilon_f}\right)^{\text{STRNEXP}} = K_1 \left(\frac{1}{\varepsilon_f}\right)^{K_2} \qquad (20)$$

where:

$K_1 = N_i \, \varepsilon_i^{K_2}$ for any point N_i, ε_i on the log-log failure plot (values of N_i and ε_i not logarithmic).

K_2 = Inverse of slope of line on a log-log plot of strain level versus cycles to failure.

ε_f = Radial strain.

A typical plot of the log of initial strain versus the log of the number of load cycles at which failure occurred for a number of similar specimens tested at different constant bending stress is shown in Figure 6.

As can be seen:

$$\log \varepsilon_f = \log \varepsilon_i - s \, (\log N_f - \log N_i)$$

$$= \log \varepsilon_i - s \, \log \left(\frac{N_f}{N_i} \right)$$

$$= \log \varepsilon_i - \log \left(\frac{N_f}{N_i} \right)^s = \log \left[\frac{\varepsilon_i}{\left(\frac{N_f}{N_i} \right)^s} \right] \quad (21)$$

$$\varepsilon_f = \frac{\varepsilon_i}{\left(\frac{N_f}{N_i} \right)^s} \quad \text{and} \quad \left(\frac{N_f}{N_i} \right)^s = \frac{\varepsilon_i}{\varepsilon_f} \quad (22)$$

$$N_f = N_i \left(\frac{\varepsilon_i}{\varepsilon_f} \right)^{\frac{1}{s}} = N_i \, \varepsilon_i^{\frac{1}{s}} \left(\frac{1}{\varepsilon_f} \right)^{\frac{1}{s}} \quad (23)$$

$$N_f = K_1 \left(\frac{1}{\varepsilon_f} \right)^{K_2} \quad (24)$$

Where: $K_1 = N_i \, \varepsilon_i^{K_2}$ and $K_2 = \frac{1}{s}$

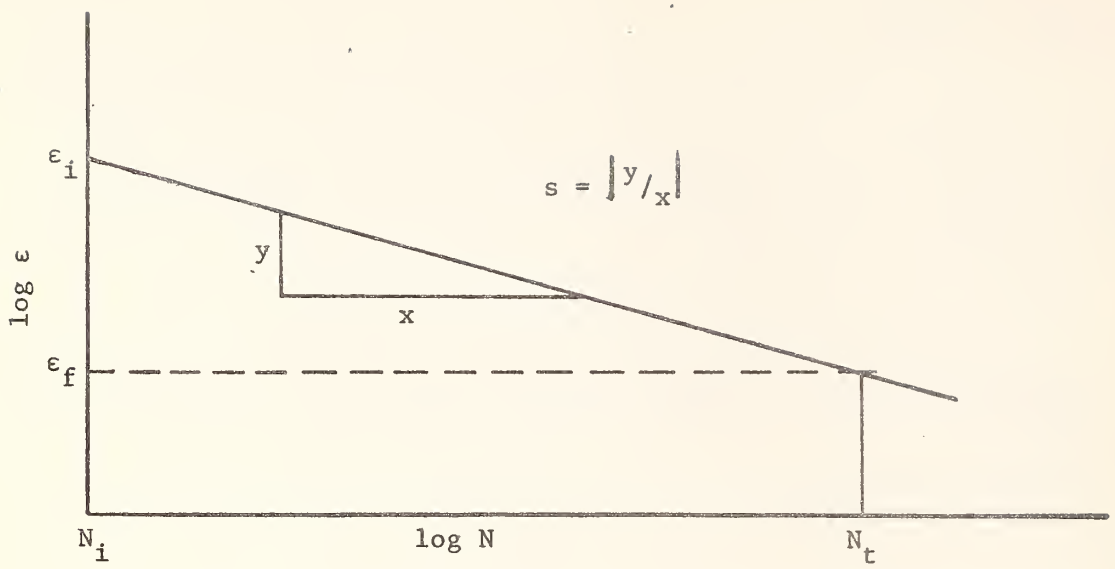

Figure 6. Typical log-log plot of initial strain versus load cycles to failure

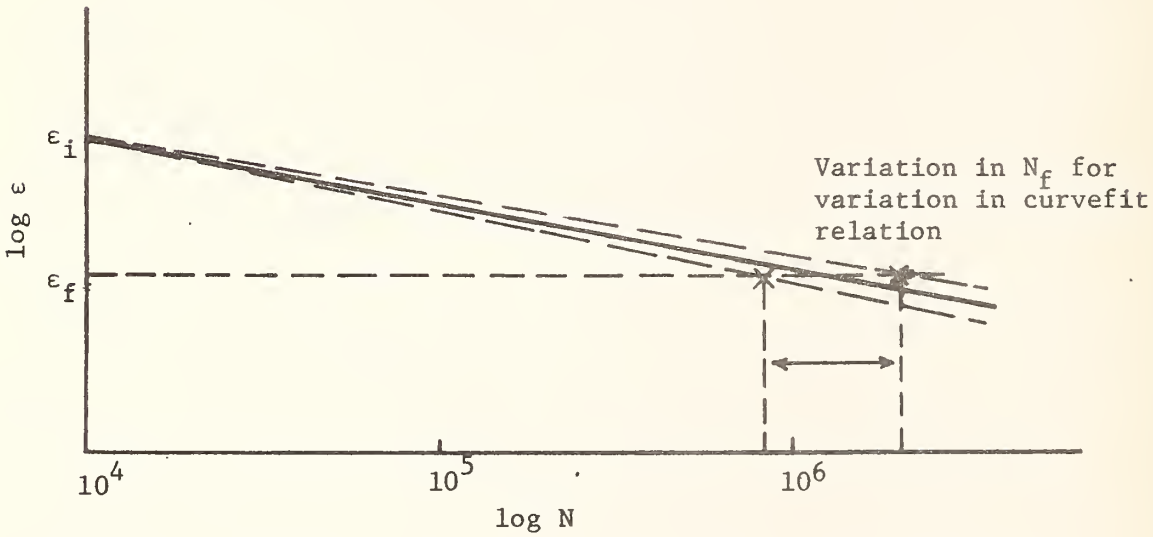

Figure 7. Graphical display of sensitivity of predicted load cycles to failure N_f to values of K_1 and K_2.

The curve fit approach is somewhat ill conditioned as the prediction is from the intercept on a log-log plot down a relatively flat line to a region on the log-log plot where a relatively minor lack of fit (variation of a predicted point from the true point on the line) can cause gross differences in the value of N_f on the log scale for a specified level of ε_f (See Figure 7 for a graphical illustration). However, no better means has yet been developed.

The Damage Index or fraction and sections cracked are calculated using Miner's damage formulation and probability theory as previously described.

Both K_1 and K_2 are dependent on temperature and relations are described in a subsequent chapter for their calculation. Each variable is input as an array with one calculated value correlating to each temperature in the "TEMPS" array.

Control Variables

In the category of control variables both data description parameters and program control options are included. Data description informs the program of the number of elements to expect in certain array, also the units in which TRANDOM is coded. Control options select the types of analysis to be performed. The following keywords can be considered control variables.

Program Control	Data Description
DEBUG	NDELTAS
ENDØFJØB	NRPØINTS
ENDØFRUN	NTCURVE
INDEX	NTEMPS
ITYPES	NTRANDØM
NØPART	NTSTATIC
PUNCH	TUNITS
TITLE	
TYPE	
UNLOAD	

Each of these keywords is explained in the Input Dictionary (Appendix E).

CHAPTER IV

SOLUTION OF PROBLEMS ENCOUNTERED DURING IMPLEMENTATION

A number of difficulties were solved during the course of the project by ARE and FHWA personnel working together. These may be broadly divided into programming incompatibilities, mathematical errors, numerical errors, engineering deficiencies and conceptual shortcomings in the model. The most important problems which were solved and the means for solving them are discussed in this chapter.

Programming Incompatabilities

Several violations of the FORTRAN language standards were found and corrected during the process of transporting VESYS to run on CDC computers. A much more difficult problem which was encountered at that time was the use of the IBM system subroutine ERRSET to suppress numeric overflow and underflow interrupts. This type of hardware dependency seriously restricts the utility of the program by limiting its use to one computer manufacturer's equipment. Until this problem was solved, the program would not run on the CDC 6600. Checks for data overflow were programmed into the code at places where interruptions were occurring. Further changes were made to eliminate cases of exponent underflow, making the program much more generally useable.

Mathematical Errors

Errors in the stochastic equations for damage index and present serviceability index were discovered and corrected. The equations involved are Taylor series approximations for the Miner's Law formulation for cracking damage and the AASHO serviceability model. The correct approximating functions were given in Chapter II. These errors were:

1. The damage index model had an incorrectly evaluated second partial derivative. In equation (29) the term $\frac{\partial^2 f}{\partial K_1^2}$, which equals $\frac{E[\varepsilon]^{E[K_2]}}{E[K_1]^3}$, was incorrectly written with a minus sign (the first partial derivative should be negative, but not the second).

2. In the AASHO equation, the rutting term $E[RD^2]$ was incorrectly expanded to:

$$E[RD]^2 + 2\ Var[RD]$$

instead of:

$$E[RD]^2 + Var[RD]$$

which is the correct expression.

3. In the slope variance term, the second partial derivative for the natural log was evaluated instead of that for the common log:

$$\frac{\partial^2 \ln[1 + SV]}{\partial SV^2} = - \frac{1}{(1 + SV)^2}$$

$$\frac{\partial^2 \log_{10}[1 + SV]}{\partial SV^2} = - \frac{\log_{10} e}{(1 + SV)^2}$$

This error affected both the mean of serviceability and its variance.

These small mistakes had an effect on the model's response, even causing negative damage indices to be printed out in some cases. The rest of the stochastic formulations in the repeated load subprogram have been checked, and that part of the model can now be employed with a considerably higher degree of confidence than before.

Numerical Curve-Fit Errors

During the preliminary runs, it became apparent that the Dirichlet curve-fit for the asphaltic concrete creep compliance data did not adequately approximate the values which were input. Alternating signs on the coefficients in the Dirichlet series were causing oscillations in the fitted curve. A printout of the residuals as a percentage of the observed values was added to the program to aid in investigating this problem. The printout revealed that some curves whose coefficients did not exhibit alternating signs differed from the observed values by as much as 25%.

In an attempt to improve the curve-fit, the effects of varying the times at which creep compliance is specified were studied for several representative curves. The number of observations given per order of magnitude of time was varied from one to four. Different spacings of the data were attempted. The effects of inputting extra observations at points where the curve-fit deviated the most were also studied. Best results were obtained with two data points per order of magnitude, evenly spaced on the log scale of time. However, none of these techniques adequately solved the problems of undue oscillations or unacceptably large residuals. It became apparent that in some cases it

would be necessary to override the program's selection of DELTAS to be used in the Dirichlet series with values which could be input explicitly.

Further modifications were made to the program in order to provide this facility. By inputting his own DELTAS, it is possible for a user of the program to significantly improve the curve-fit of the compliance data. A series of Type 4 runs (curve-fitting analysis only) should be run using different sets of DELTAS in order to select the set which best approximates the creep compliance curve. In selecting the sets of DELTAS to be tried, the user should be guided by the following general observations:

1. Supply two time points per order of magnitude as suggested above. A good set of values to use for TSTATIC is: .001, .003, .01, .03, .1, .3, 1, 3, 10, 30, 100.

2. Approximately one δ should be provided for every order of magnitude over which time varies in the TSTATIC array. Too many δ's will cause excessive oscillation in the fitted curve as indicated by alternating signs in the coefficients produced, but too few δ's will result in a poor fit indicated by large residuals in the printout.

3. The technique which seems to work best is an iterative one. First, the data are curve-fit using the δ's provided by the program. From the printout of the residuals, it is possible to identify the time interval in which the fit is poorest. An extra δ is then provided to improve the fit in the interval which has been identified. The other δ's must be spaced apart somewhat in order to prevent oscillation which might be caused by the close proximity of neighboring δ's.

4. In order to select a δ for a certain time interval, it is necessary to know what will be the range of its effects. A useful rule of thumb is that a δ will have its influence on the fitted curve over the time range from $.05/\delta$ to $3.0/\delta$. If the predicted values differ from the observed curve most at a time t, then a δ whose value is $.69/t$ will have its range of influence centered about the region of the curve where the fit must be improved.

Variation of Fatigue Relationships for Different Temperatures

VESYS IIM included a parabolic relationship for automatically varying the fatigue coefficient $K1_i$ and exponent $K2_i$ with changes in

temperature. The relationship was based on laboratory beam tests conducted by the Asphalt Institute (Ref 6)[1].

It became apparent during material characterizations for the sensitivity analyses of VESYS IIM that some better means of revising K_{1i} and K_{2i} with temperature was required. Since the data from various sources varied greatly, normalized multipliers $K_1(T)/K_1(70°F)$ and $K_2(T)/K_2(70°F)$ were used to allow meaningful comparison of the variation of K_{1i} and K_{2i} with temperature. The development of normalized multipliers for tentative use pending further research is described in detail in Appendix D.

The fatigue response variations with changing temperature differed so much for different mixes that the relationships developed for revising K_{1i} and K_{2i}, while adequate for this sensitivity analysis, must be considered as approximate for K_{2i} and representative of only an average condition for K_{1i} for more general use. As K_{1i} and K_{2i} are negatively correlated to a relatively high degree, it might be possible to emphasize the change in K_{2i} in future research and calculate K_{1i} from some general correlation curve.

Unrealistically High Values of Creep Compliance

The creep compliance relationships with duration of loading available from a literature search were found to generally represent low values of material stiffness (inverse of creep compliance). Their use resulted in calculation of excessive strains with the elastic layer subroutine in VESYS IIM. This in turn resulted in very unrealistic cracking and rutting predictions.

It was reasoned that a creep compliance-load duration array must approximate the inverse of the dynamic or resilient modulus for the same material at the same load duration and temperature as used in the test to be a reasonable characterization of the in-service pavement material. A procedure for obtaining creep compliance arrays from standard creep compliance tests after the sample has been dynamically conditioned was developed and is described in detail in Appendix B. The results are reduced magnitudes for creep compliance more consistent with the inverse of the dynamic modulus and believed to be a greatly improved characterization of the true material response.

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[1] Sharma, M. G. and W. J. Kenis, "Evaluation of Flexible Pavement Design Methodology", paper presented at the Annual Meeting of the Association of Asphalt Paving Technologists, Phoenix, Arizona, February, 1975.

Modifications to Pavement Life Model

During the preliminary sensitivity runs, erratic variations were observed in the expected life predictions. Frequently, the predicted life appeared to disagree with the serviceability curve. As this condition might have been caused by an error in the code, the problem was investigated. This variability could be traced to inadequacies in the numerical integration technique, which was used to calculate expected life. Further analysis pointed to the conclusion that expected life itself was a misleading statistic, in the engineering sense, thus it was replaced with a more meaningful indication of pavement service life.

Expected Life Model

The model previously employed was quite straightforward. (See the detailed mathematical derivation in Appendix A). If time-to-failure is considered to be a random variable, then the expected value of this variable is simply the mean of its probability density function. Under certain reasonable assumptions, it is possible to approximate this mean by the integral from zero to infinity of the pavement's reliability function. This function is estimated in the program on the basis of the calculated means and variances of serviceability throughout the period of the analysis. Since estimates of reliability are only obtained at the times specified for repeated load calculations by the user of the program, the integral from zero to infinity is approximated in the program by the integral from zero to the last time specified for the analysis. As might be expected, changing the last value input for TRANDOM while holding all other variables constant can produce gross variations in expected life predictions without in any way altering the characteristics of the pavement being modeled. Furthermore, it can be shown that the computational error which is committed by omitting the infinite tail of this integral is unacceptably large for some realistic pavement modeling situations (See Appendix A).

Various techniques could be used to achieve a better approximation for this integral. However, the distribution of time-to-failure is such that its mean will tend to be a very large value. So large in fact, that the expected value of pavement serviceability will have reached the specified failure level long before the expected lifetime has run out (See Appendix A).

Reliability and Service Life

As the mean time-to-failure is unlikely to be achieved by more than a small fraction of pavement sections constructed, it is necessary to find a more meaningful statistic to describe the service life of a

pavement design. The solution to this problem which has been adopted is based on the concept of pavement reliability. This term is used here in the sense of the following definition: "Reliability is the probability that serviceability will be maintained at adequate levels from the user's point of view, throughout the design life of the facility" (Ref 7)[1]. The minimum acceptable reliability varies with the type of pavement being designed. For example, a heavily traveled urban highway would ordinarily need to be designed with a higher reliability than that required for a farm to market road. As pavement reliability can be expected to decrease with time, the service life predictions will depend in part on the minimum reliability which will be tolerated by the designer of the facility. For this reason, a new input variable named TOLERNCE has been added to the program. Service life is predicted by estimating the time at which reliability reaches the specified minimum level. If this minimum is not reached within the period specified for the analysis, service life will be predicted on the basis of extrapolation (See Appendix A for a description of technique used). In this case, a message is printed warning the user that the estimate may be inaccurate and suggesting that the problem be rerun using a longer analysis period if accurate service life predictions are required.

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[1] Lerner, A. C. and F. Moavenzadeh, "Reliability of Highway Pavements", Highway Research Record No. 337, Highway Research Board, 1970.

CHAPTER V

DEVELOPMENT OF REALISTIC INPUT DATA FOR THE SENSITIVITY ANALYSIS

The effort reported in this chapter was one of the most difficult tasks accomplished in this research project. Some of the inputs are new to the engineering profession and others were stochastic variations of known variables for which few data were available.

The input values used appear in Table 1. Where an independent variable or factor was varied in both the cracking and roughness factorials, only low and high level values were used. If it was not varied in one of the factorials but was in the other, the mean value was also used when it was held constant. If the factor was constant for both factorials, only the mean value appears. For some factors used as arrays of values, Table 1 refers to other tables or pages where these arrays may be found.

The goal for selection of all the independent variables to be studied was to arrive at a mean value, $\bar{x}$, representative of the variable's distribution over the range of its "between project" or overall variation. (Stochastic inputs such as variance or coefficient of variation for a variable were based on "in-project" variation as they represent variability for a specific value of a factor rather than of a number of possible values). Once a mean value was established, the standard deviation, σ_x, was identified. Where the variable's values might be expected to be normally distributed, the low level value was selected as $\bar{x} - \sigma_x$ and the high level as $\bar{x} + \sigma_x$. If the variable had a skewed distribution and the standard deviation would have given unrealistic levels, the values at either side of the central 68% of the distribution were selected to approximate a span of one standard deviation either side of the mean.

There were not always sufficient data to offer a large enough sample for meaningful statistical analysis. In these cases, engineering judgment and experience were applied to extrapolate from the limited data in order to obtain the best possible values to use.

Traffic and Wheel Load Data

The intent in developing a value of LAMBDA to represent truck traffic was to arrive at reasonable values for an urban freeway or rural interstate highway having more or less median traffic compared to the whole range from rural roads to heavily travelled urban freeways.

Table 1. Input data for main sensitivity analysis for VESYS II (M)

Independent Variables	Magnitude Levels Low	Mean	High
ALPHA(1)	0.68	0.75	0.82
ALPHA(2)	-	0.88	-
ALPHA(3)	0.69	-	0.94
AMPLITUD	58	-	95
BETA	-	0.054	-
COEFK1	0.30	-	1.24
COEFK2	0.04	-	0.1
CORLCOEF	-	1.0	-
CORLEXP	0.044		0.072
DELTAS		(100,30,8,1,0.1, 0.03,0)	
DURATION	-	0.0125	-
GNU(1)	0.20	0.40	0.60
GNU(2)	-	0.25	-
GNU(3)	-	0.045	-
ITYPES		100100	
K1K2CORL	-	-0.867	-
LAMBDA	2000	-	4500
LAYER1	see Appendix B, Figure 38		
LAYER2*	4.55×10^{-5}, 6.25×10^{-5}		
LAYER3*	1.11×10^{-4}, 2.0×10^{-4}		
NDELTAS		7	
NTEMPS		12	
NTRANDOM		12	

Table 1. Input data for main sensitivity analysis for VESYS II (M) (con't)

Independent Variables	Magnitude Levels Low	Mean	High
NTSTATIC		11	
PSIFAIL		2.5	
QUALITY0		4.2	
RADIUS	-	6.4	-
REFTEMP	-	7.0	-
STDEV0	-	.29	-
STRNCOEF		See Table 18, Appendix D	
STRNEXP		See Table 18, Appendix D	
TEMPS		See Table 18, Appendix D	
THICK1	3.5	-	5.0
THICK2	15	18	21
TOLERNCE	-	1 through 12	-
TUNITS		6	
TYPE		1	
VARCOEF1	0.1	0.2	0.3
VARCOEF2	0.1	0.2	0.3
VARCOEF3	0.25	0.325	0.40
VCAMP	196	342	529
VCDUR	-	5.4×10^{-6}	-

* Single-valued creep compliance, constant with time.

A Texas study involving rural and urban traffic counts included six sections in various larger cities. The average daily traffic ranged from 2820 to 125,542 vehicles in both directions. A section in San Antonio having four lanes and an ADT of 49,206 was considered typical.

In 1963, 12.6% of all traffic was truck traffic according to Ref 8[1]. Table 5.26, p.202 of Ref 9[2] establishes truck traffic in urban areas as 15% of all traffic. Ref 10[3] page 64 indicates a lane distribution factor of 80-100% for a four-lane highway, so 90% was selected.

Daily truck traffic was calculated as follows:

$$\text{Daily truck traffic} = \frac{49,206}{2} \times .9 \times .15 = 3321$$

Considering an average daily truck traffic value of around 3300 as desirable for this study, a high of 4500 tpd and a low of 2000 tpd was judged to represent a range consistent with the concept of one standard deviation on each side of a mean.

LAMBDA was input as trucks per day as defined in VESYS documentation. Unfortunately, VESYS IIM makes no correction to modify trucks to axles per day so this error in documentation was noticed too late for correction to axles per day prior to the main sensitivity analysis. As a result, it is felt that the range selected (2000 to 4500) for this variable is more representative of a rural state highway or less travelled interstate highway with say 500 to 1200 trucks per day than the typical rural interstate highway intended. The effect on the sensitivity analysis was likely to somewhat underestimate the importance of LAMBDA. If input had been in axles per day, LAMBDA might have approached AMPLITUD in significance.

Due to the use of a fixed value of radius over the range of wheel loads and tire pressures for a specific problem, the variable AMPLITUD must include more than just tire pressure as it also provides wheel load information. It was necessary as a beginning to obtain some typical wheel load distribution. The one used was measured during a South Dakota study, reported in Reference 11[4] and plotted in Figure 8. Notice the skewed distribution. Using the extremes of the central 68%

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[1] "A Policy of Geometric Design of Rural Highways", American Association of State Highway Officials, 1965.

[2] "Traffic Engineering Handbook".

[3] AASHO Interim Guide.

[4] "1970 Truck Weight Study", South Dakota Department of Highways, Research and Planning Division, in cooperation with U.S. Department of Transportation, Federal Highway Administration.

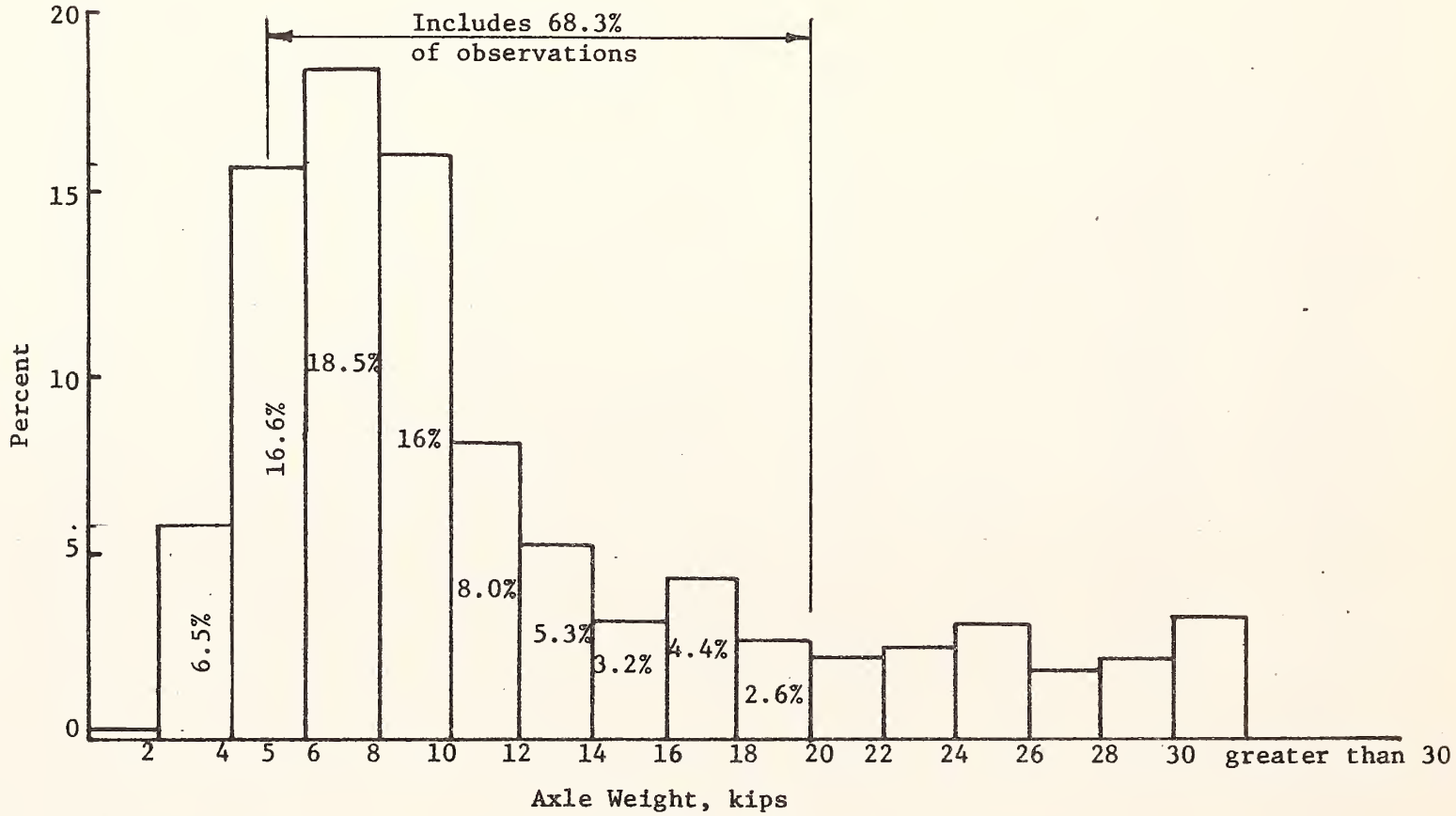

Figure 8. Histogram for axle loads, South Dakota study in 1970.

of the distribution provides a low level of axle loads of 5,000 lbs. and a high of 20,000 lbs. The wheel loads would be 2,500 and 10,000 lbs. for development of low and high values of AMPLITUD. The mean value for wheel load is 4,500 lbs.

Using three levels of tire pressure (70, 80, and 90 psi), values of radii may be calculated for various wheel loads from 2500 lbs. to 9500 lbs. These calculated radii appear in Table 2. Using the relationship

$$\text{AMPLITUD} = \frac{\text{Wheel Load}}{\pi(\text{RADIUS})^2} ,$$

values of AMPLITUD may be calculated for consistent levels of wheel load and average loaded radius from Table 2. These values were calculated using average values of radius (shown in parentheses in Table 1) and appear in Table 3. By weighting the values in Table 3 directly with percentages of load distribution in each wheel load increment from Figure 8 and indirectly by tire pressure, the distribution of AMPLITUD shown in Figure 9 may be obtained.

As AMPLITUD is not normally distributed, the central 68% was used as shown to obtain a low level of 58 psi and a high level of 95 psi.

As the variance of AMPLITUD called VCAMP was not found in the preliminary sensitivity analysis to be significant for the roughness factorial and only mildly significant for the cracking factorial, substantial effort was not warranted for obtaining accurate values. Furthermore, there was no real source for doing so. As the range for two standard deviations in AMPLITUD was taken from Figure 9 to be 37, a standard deviation might be approximated as 18.5 psi. The variance would then be the square of 18.5 or 342. On the basis of judgement, it was decided that the standard deviation might vary from 14 to 23 psi, so a low for VCAMP of 196 and a high of 529 resulted. As expected, VCAMP was found to be insignificant for cracking by the main sensitivity analysis, so the values used had no measurable effect on the results.

The DURATION of the wheel loading at a point was calculated for the mid-depth of the first layer on the assumption that the loaded area spreads outward with depth along a 45° vertical line. The equation for the duration of load at a point at the surface is:

$$\text{DURATION} = \frac{2(\text{RADIUS})}{(\text{Mean Truck Speed})}$$

$$\text{DURATION} = \frac{2(\text{RADIUS})}{17.6(\text{Mean Truck Speed in mph})} \text{ In/Sec.}$$

At the midpoint of the first layer:

$$\text{DURATION} = \frac{2\ (\text{RADIUS} + \tfrac{1}{2}\ \text{THICK1})}{17.6\ (\text{Mean Truck Speed in mph})} \text{In/Sec.}$$

Table 2. Radii or values for RADIUS calculated for a range of tire pressures and wheel loads

Wheel Load (lbs)	Tire Pressure in psi 70	80	90
2500	3.37	3.15	2.97
3500	3.99	3.73	3.56
4500	4.52	4.23	3.99
5500	5.00	4.67	4.41
6500	5.4	5.08	4.79
7500	5.8	5.46	5.15
8500	6.22	5.82	5.48
9500	6.5	6.15	5.8

Table 3. AMPLITUDE values as functions of wheel load and RADIUS

Wheel Loads (lbs.)	Loaded Radius, Inches						
	3-3.5 (3.25)	3.5-4 (3.75)	4-4.5 (4.25)	4.5-5 (4.75)	5-5.5 (5.25)	5.5-6 (5.75)	6-6.5 (6.25)
2000	75						
3500		58					
4500			80	63			
5500			98	78	64		
6500				92	75		
7500					87	72	
8500					98	82	69
9500						92	77

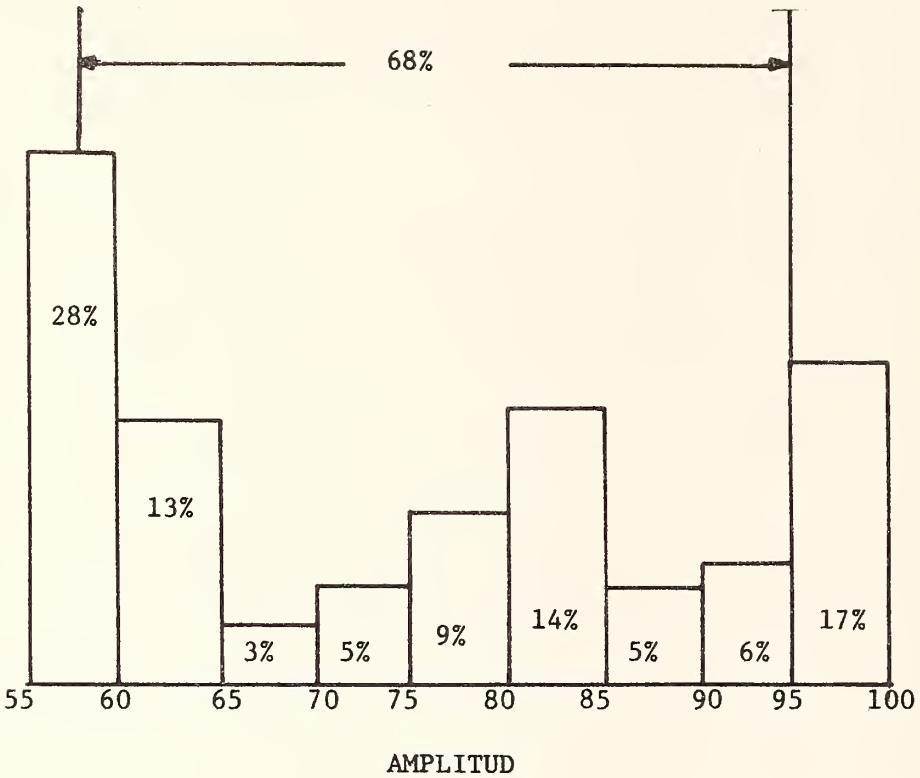

Figure 9. Distribution of AMPLITUD weighted by traffic and tire pressure.

Using an assumed THICK1 of four inches, five values of RADIUS from 3.5 to 5.3 and three truck speeds of 50, 60 and 70 mph; an array of values for DURATION were generated with a mean of 0.0125 seconds. The standard deviation was 0.00233 seconds and thus the variance VCDUR was 5.4×10^{-6} seconds. This approach was believed to give rational values of DURATION and VCDUR. Neither variable was found to be significant during the preliminary sensitivity analysis so the actual values used were not especially important as long as they were reasonable.

Materials Characterizations

The studies required to establish reasonable values for the permanent deformation, creep compliance and fatigue life potential characteristics of the pavement materials were so broad and their results so detailed to report that they have been reported separately in Appendices B, C and D, respectively. A breakdown of the variables by appendix in which their values are developed appears in Table 4.

The rationale for use of the value of 1.0 for CORLCOEF appears in its description in Appendix E.

Pavement System Data

The wheel load, traffic and materials data described previously were used to design typical pavement sections that were compatible with the means plus or minus a standard deviation. Design was based on Texas Highway Department procedures with some consideration of other methods. This defined the low and high levels of THICK1 and THICK2 as shown in Table 1.

The initial Present Serviceability Index QUALITYO was established as 4.2, because that is about what may be produced with current construction technology. The standard deviation STDEVO for QUALITYO was selected to be 0.29 based on data in Reference 12[1]. The failure level of Present Serviceability Index called PSIFAIL was selected as 2.5 because that level has been generally defined as the minimum level considered acceptable by the traveling public.

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[1]Hudson, W. R., "State-of-the-Art Predicting Pavement Reliability from Input Variability", a report prepared for the U.S. Army Waterways Experiment Station, Vicksburg, Miss., December 1974.

Table 4. Appendices where development of reasonable values for specific variables are described.

Characterization	Appendix	Variable
Creep Compliance	B	BETA
		LAYER1
		LAYER2
		LAYER3
		VARCOEF1
		VARCOEF2
		VARCOEF3
Permanent Deformations	C	ALPHA(1)
		ALPHA(2)
		ALPHA(3)
		GNU(1)
		GNU(2)
		GNU(3)
Fatigue Life Potential	D	COEFK1
		COEFK2
		K1K2CORL
		STRNCOEF
		STRNEXP
		TEMPS

Realistic Values for CORLEXP

In the VESYS IIM program, Equation (1) is used to obtain slope variance on the basis of the stochastic variation of rut depth. For clearer understanding, Equation (1) may be approximated by the following simplified relation:

$$E[SV] \simeq \frac{2B}{C^2} \text{Var}[RD] \tag{25}$$

where:

E[SV] = mean slope variance in radians x 10^6,

Var[RD] = rut depth variance in inches^2,

B = CORLCOEF, and

C = CORLEXP

For reasons outlined in Appendix E, it is reasonable to set the value of CORLCOEF to one in all cases. With this simplification it is possible to treat equation (25) as a simple linear regression of E[SV] on Var[RD] with intercept forced through the origin.

STEP01, "A Statistical Computer Program For Stepwise Multiple Regression", was used for four samples of E[SV] and Var[RD] data which were tabulated on page 46 of Reference 13[1]. The combined data from the four samples were also analyzed separately. The following results were obtained:

Section	Regression coefficient	R^2 for regression	Value for CORLEXP
Illinois	380	.870	.073
Indiana	1058	.601	.043
Minnesota	745	.845	.052
AASHO	406	.762	.070
Combined	578	.605	.059

The value listed for CORLEXP is based on the assumption that CORLCOEF equals one, yielding the equation:

$$C = \sqrt{\frac{2}{K}} \tag{26}$$

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[1]Carey, W. N., Jr., Irick, P. E., The Pavement Serviceability - Performance Concept, HRB Bulletin 250, 1960.

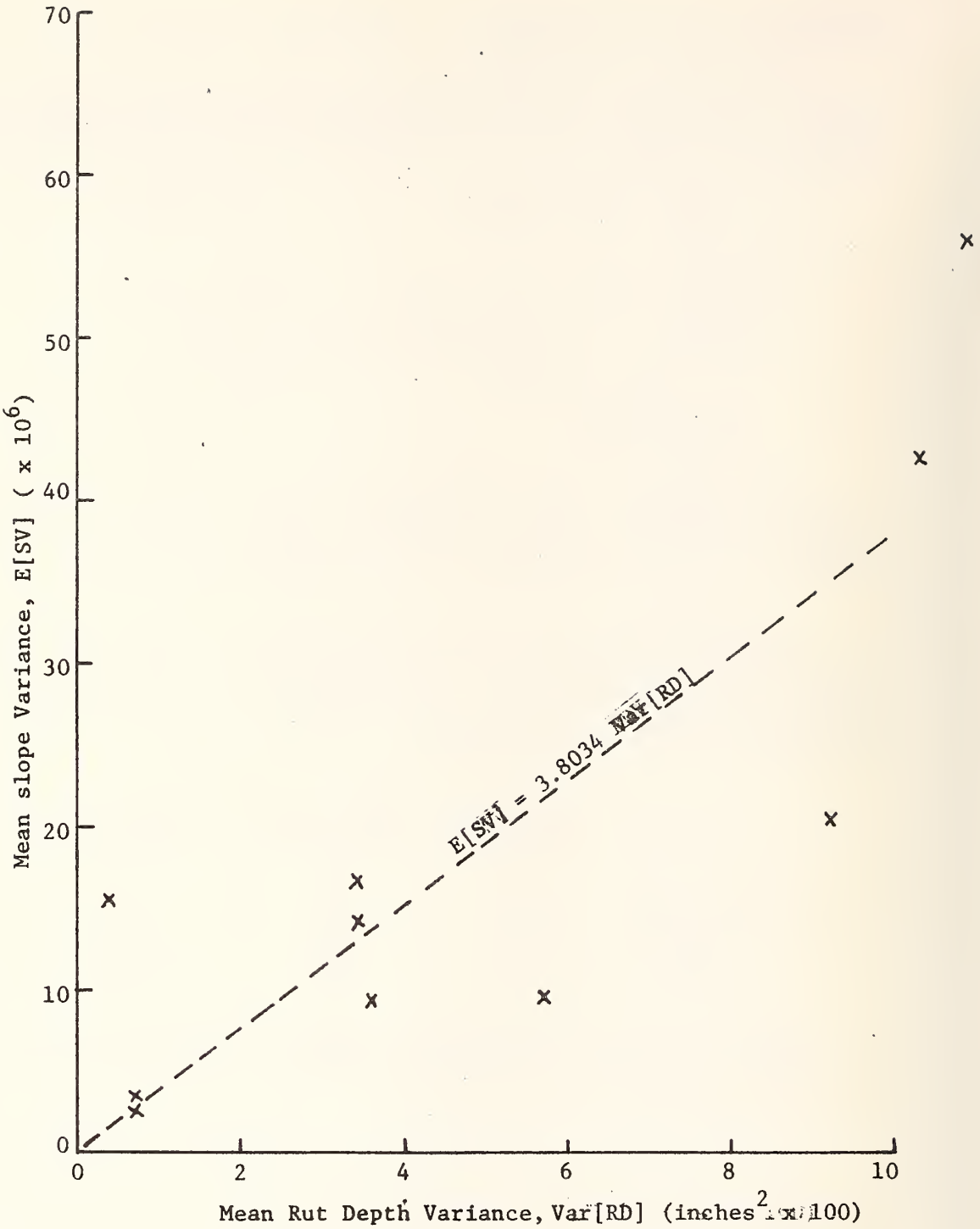

Figure 10. Mean slope variance versus mean rut depth variance for Illinois sections.

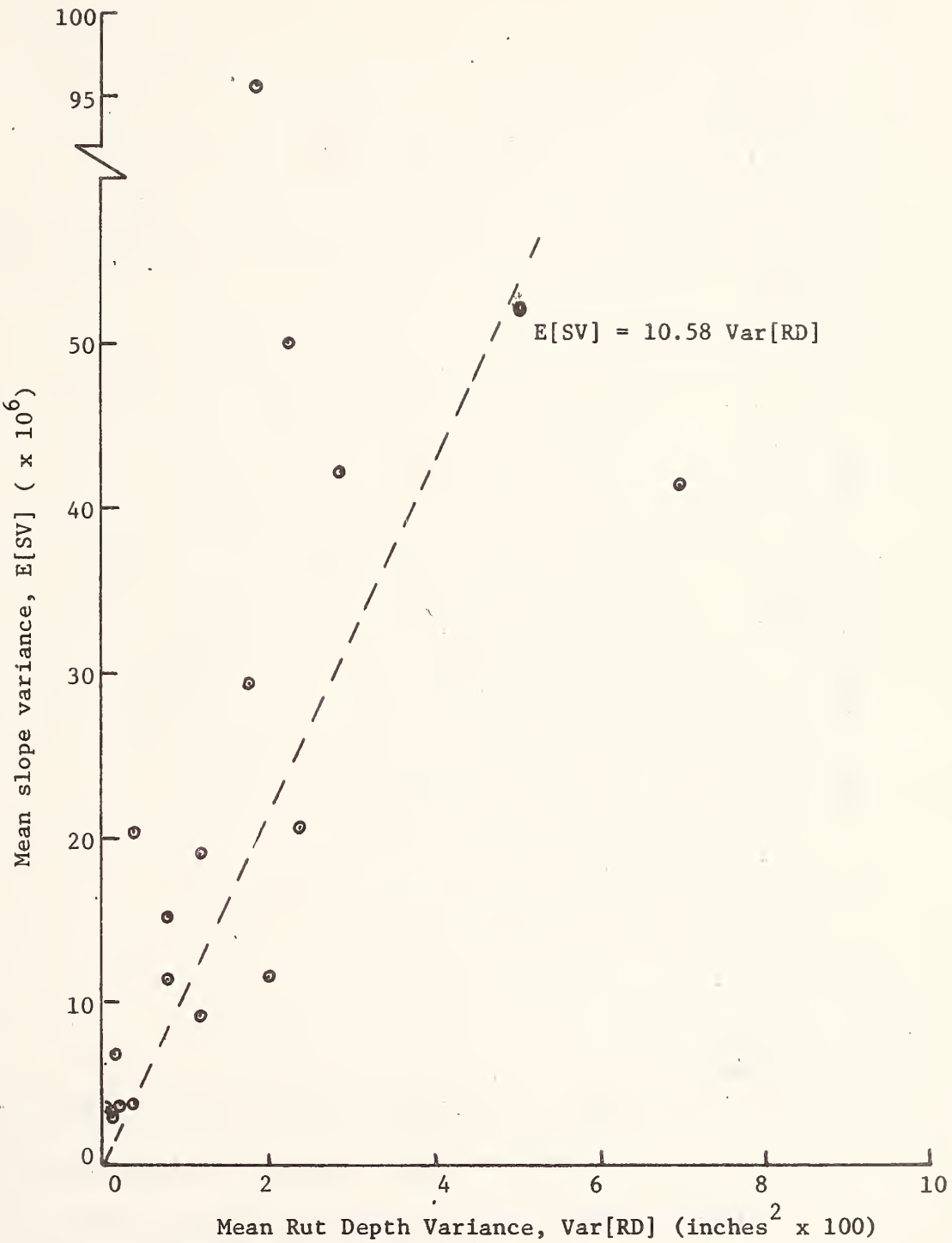

Figure 11. Mean slope variance versus mean rut depth variance for Indiana sections.

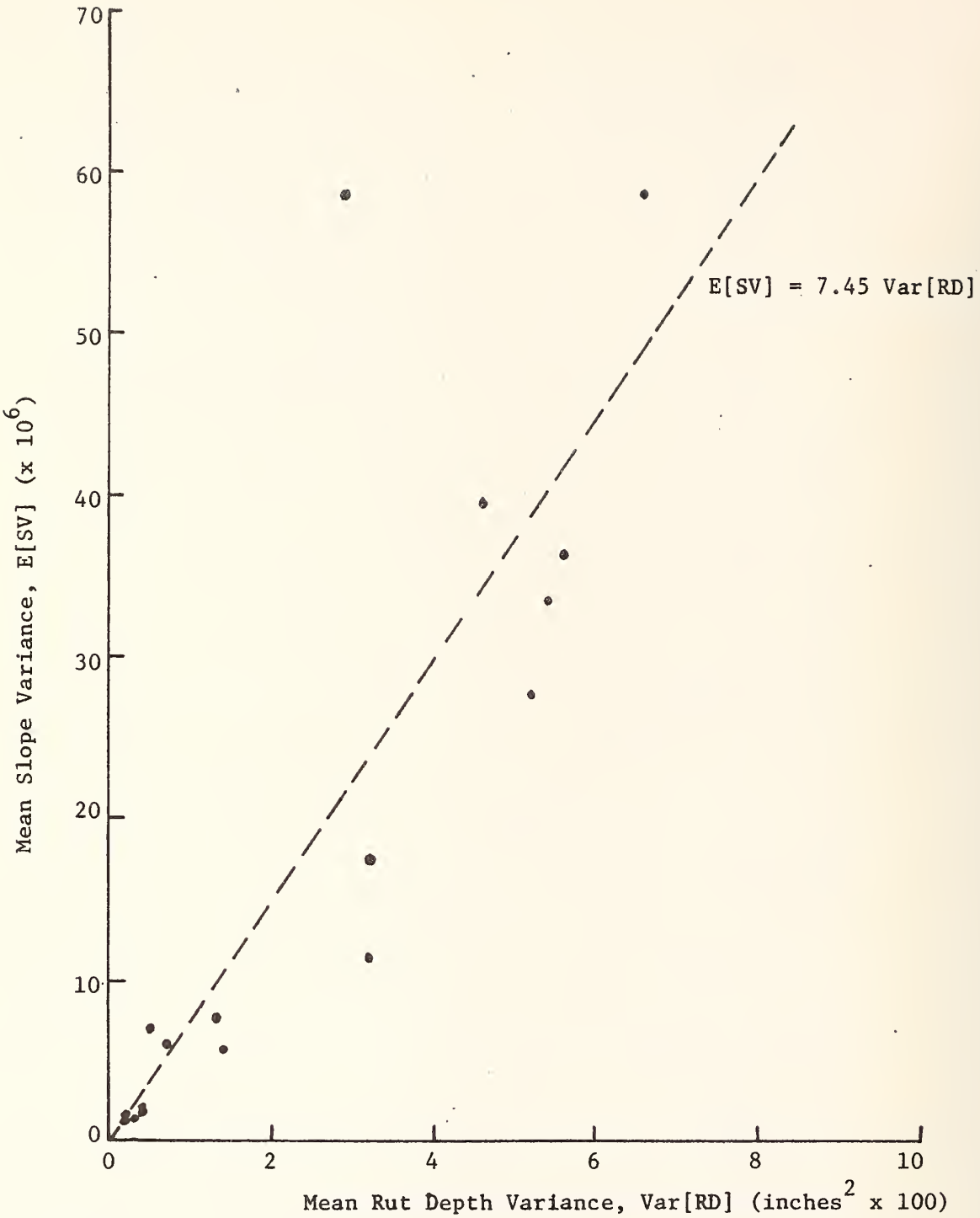

Figure 12. Mean slope variance versus mean rut depth variance for Minnesota sections.

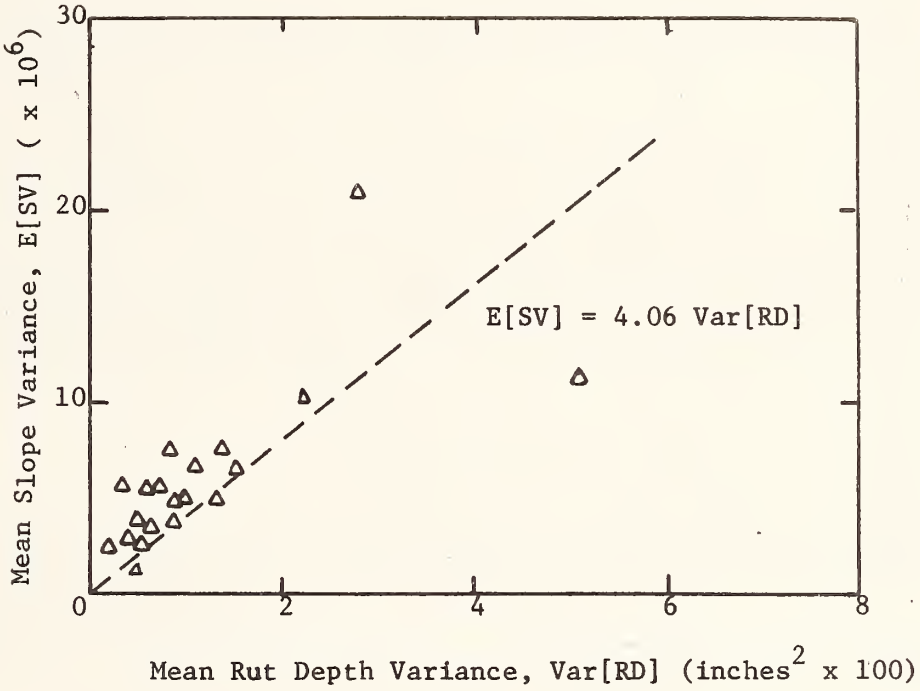

Figure 13. Mean slope variance versus mean rut depth variance for AASHO test sections.

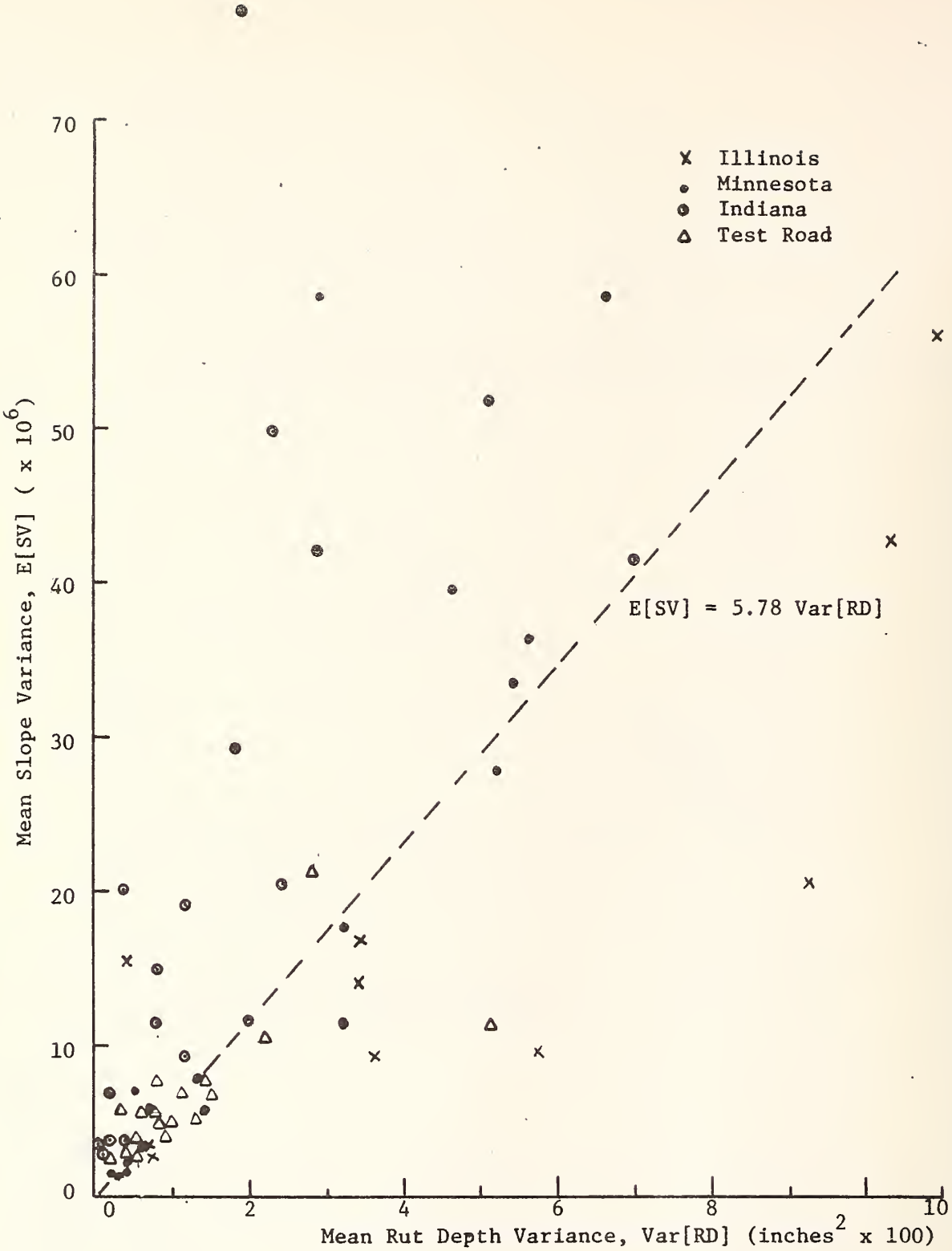

Figure 14. Mean slope variance versus mean rut depth variance for all data combined.

where:

C = CORLEXP, and

K = the regression coefficient.

Equation (26) follows immediately from the fact that the regression coefficient is equal to the expression $2B/C^2$ in equation (25).

The mean value of CORLEXP obtained from the four sections is .05949, with a coefficient of variation of 24%. The median value of .058 was used instead of the mean.

Plots of E[SV] versus Var[RD] for the five samples are shown in Figures 10 through 14. Inspection of the scatter suggests that a quadratic term in Var[RD] would improve the fit somewhat. This could be investigated, and useful recommendations for improving the VESYS IIM model might be found. It should be noted that slope variance is the most significant factor in the AASHO equation for serviceability. Consequently, the value input for CORLEXP has a considerable effect on the program's PSI predictions.

Control Variables

There is very little choice in values for control variables once certain decisions are made relative to other input variables and type of solution to be run. These variables are all described in Appendix E and are listed below:

DELTAS	REFTEMP
ITYPES	TOLERNCE
NDELTAS	TRANDOM
NTEMPS	TUNITS
NTRANDOM	TYPE
NTSTATIC	

CHAPTER VI

COMPARISONS OF VESYS IIM PREDICTED TO MEASURED PERFORMANCE

One of the principal objectives of this research program was to see how well VESYS IIM at its current state of development could predict performance of actual pavements. This requires both a detailed historical record of the pavement's performance and sufficient information to provide the detailed input required by VESYS II M. While no pavement section exists with all the information required, the best documented available appeared to be the Brampton Test Road and certain sections from the AASHO Road Test.

The Brampton Road Test was conducted in southern Ontario, Canada and records have been kept from 1965 until the present. More advanced information was available for these materials used on this test as it was conducted more recently and they had undergone considerable dynamic testing at the University of Waterloo and by the Asphalt Institute.

The AASHO Road Test was completed in 1962, but many sections of highway included were overlaid and opened to traffic. Measurements of rut depth and cracking have continued from 1962 to 1974. As this test was conducted prior to extensive use of dynamic and creep compliance testing for pavement materials, such data were generally unavailable for the materials used and had to be developed.

As the necessary information was more readily available for the Brampton Road Test, the analytical work and comparisons were conducted for it first. Initial performance predictions were poor, so study was continued with different combinations of input data derived partially from testing of Brampton materials and from other sources until more reasonable predictions were obtained.

This both emphasized the importance of accurate input variables to VESYS IIM and indicated that VESYS IIM could produce reasonable predictions for sets of input variables with rational values.

Fortunately, ARE was engaged in another separate research effort for FHWA and had obtained cores of the asphaltic concrete, bulk samples of base and subbase materials and undisturbed subgrade samples from a number of pavement sections included in the AASHO Road test. Thus samples were available for creep compliance and other dynamic testing for direct characterization of the materials through specific tests to service VESYS IIM input requirements. The comparisons for AASHO Road test sections could then represent a case where current state-of-the-art testing could be used to characterize materials and extensive geometric and traffic data were available. This was then a test of VESYS IIM involving its use by an agency under near-optimum conditions as far as knowledge of the computer program and its input requirements were concerned.

Brampton Test Road Comparisons

The Brampton Test Road studies were conducted in behalf of ARE by Dr. R. C. G. Haas, Professor, University of Waterloo, and his associate, Mr. Frank Meyer. This approach was taken because of their thorough knowledge of the Brampton Road Test and its pavement materials and difficulty in long-range coordination. A condensed version of the resulting report to ARE is included in Appendix F. This report includes description of the study, the pavement sections, input values used and conclusions by Haas and Meyer.

Input Variables

Review of the input varables used in lieu of the latest data derived from the materials studies indicates that:

1. The creep compliance curve for the Penn State Test Track is much lower (as shown in Figure 25of Appendix B) than the preponderance ofdata from standard creep compliance testing. It is, however, still somewhat higher than the curves obtained from testing after dynamic conditioning on AASHO cores as shown in Figure 37, Appendix B. The Penn State curve appears to be much more simulative of real pavements than the Brampton curve because of its relatively stiffer characterization (i.e., lower creep compliance).

2. Creep compliance values for the base and subgrade were reasonably obtained from resilient moduli test data for Brampton materials and are generally consistent with values used for the AASHO Road Test and the sensitivity analysis. LAYER3 for the subgrade represents a very stiff subgrade condition.

3. The value for BETA of 0.162 is higher than most values from standard creep compliance testing and the value of 0.085 actually developed for Brampton A.C. A mean value developed by ARE from eight different sources was about 0.11, with values ranging from 0.061 to 0.17. The value of BETA developed by ARE using dynamic conditioning before testing AASHO Road Test cores was 0.054. As VESYS IIM was found to be insensitive to changes in BETA across a range of 0.06 to 0.17, the value used should not have affected the results of the study.

4. The Penn State values for ALPHA(1) and GNU(1) of 0.765 and 0.38 respectively, would plot into Figure 66, Appendix C almost as the mean values between poor and good rutting performance. While this apparently represents a reasonable set of values to use in general, they may or may not be representative for the Brampton pavements.

5. The values for ALPHA(2) and GNU(2), except for Section 3, represented a material somewhat more prone to rutting (low ALPHA and high GNU) than those few data available and plotted in Figure 44, Appendix C. It should be noted that creep compliance should be positively correlated with GNU and negatively correlated with ALPHA. This seems to be the case for the values of LAYER2, ALPHA(2) and GNU(2) for the four sections. VESYS IIM is relatively insensitive to ALPHA(2) and GNU(2), so these values should have little effect as they are reasonable.

6. The values of ALPHA(3) and GNU(3) for the subgrade represent a fairly stiff clay per Figure 44, Appendix C. This is consistent with the low values of creep compliance (LAYER3), which represent a range of resilient moduli from 31,500 to 35,000 psi. These values are quite high for subgrades in general, but may be reasonable for the low stress levels to be found under full-depth, deep strength or thick conventional pavements.

7. The value of 0.7 used for COEFK2 was selected prior to a sufficient understanding of the variation of K_2 and its effects and would grossly increase the estimates of cracking due to an error in the stochastic formulation, Equation (4), (discussed in Chapter VII). This did not particularly matter as the erroneous formulation itself invalidates the cracking predictions unless a very low value of COEFK2 (say less than 0.04) was used.

Table 20 in Appendix F provides a summary of input data for 16 separate computer runs with source of data for the variables and pavement section identified for each run. The three-letter legend A, B and K represents ARE-developed data, Brampton Road Test data and Penn State data used by Kenis, respectively. Most of the magnitudes for input from these three sources appear in Appendix F.

Calculated Responses

Table 21 in Appendix F provides calculated output or responses after one month, at the calculated expected life or at 87 months (last point specified) for each of the 16 computer runs.

It can be seen immediately that the computed responses for Runs 2 through 6 were not practical. This is believed to have been caused by the high values of creep compliance developed for the Brampton A.C. The Penn State values used subsequently were much lower (and thus stiffer), so lower strains were calculated. As discussed in Appendix B, the usual high values from creep compliance tests without dynamic conditioning are not believed to be simulative of real pavements under repetitive loading.

Runs 7 and 8 were experiments with only traffic (LAMBDA) varied and were not intended for use in comparisons of predicted and measured performance.

The only difference between runs 6 and 9 is replacement of the Brampton LAYER1 with the Penn State LAYER1. The responses were immediately revised from the ridiculous to values in the right order of magnitude. As can be seen from Figure 85 of Appendix F, this prediction for rut depth at 70 months was still about 50% higher than measured, so additional experimentation was warranted.

Run 15 was the final run for Section 11 and the only difference from Run 9 was that ALPHA(1), GNU(1) and BETA were changed from Brampton to Kenis Penn State data. As BETA would have little to do with the results, the lower permanent deformation potential for the Penn State data appears to have reduced rutting by one third and reasonably good agreement was obtained between both rut depth and serviceability (See Figures 85 and 89 , Appendix F).

Making the same comparisons for Runs 5 and 10 (Section 4), the only important difference is again LAYER1 and the effect was the same. The calculated rut depth of 0.58 inches compares very well indeed with the approximately 0.5 inches on the measured curve shown in Figure 83 of Appendix F.

Run 11 substituted ARE values for the stochastic variables VARCOEF1, VARCOEF2, and VARCOEF3 for the creep compliance curves and replaced BETA, GNU(1) and GNU(2) with the Brampton values. The changes in responses were slight but in the right direction.

Run 12 substituted the ARE curve for LAYER1 and reinstated Brampton data for LAYER2, LAYER3, and GNU(3). This had the effect of increasing the resilient modulus of the subgrade from about 2000 psi (Penn State), to 34,000 psi. The decreased values in ARE's LAYER1 also would stiffen the pavement system, but calculated rut depth is relatively more sensitive to LAYER3 than LAYER1 as learned from the sensitivity analysis. The change in LAYER2 would have reduced the stiffness of the system and would have compensated in a small way for the effects of the changes in LAYER1 and LAYER3. The change in GNU(3) would have had little effect. The result of these changes in input values as might be expected was a severe reduction in rut depth predicted.

Run 14, the final run with Section 4, replaced the ARE creep compliance curve for LAYER1 with the Penn State curve, replaced the ARE BETA with the Penn State value and replaced the Brampton ALPHA(1) and GNU(i) values with Penn State values. Instead of increasing the prediction of rut depth, the prediction was decreased, which implies:

1. It apparently did not make much difference whether the

Penn State or ARE dynamic creep compliance curves for LAYER1 were used; or possibly the effects of a weaker LAYER1 and less permanent deformation tendency in the A.C. were compensating.

2. The primary difference was in the subgrade stiffness and the Brampton values were applicable.

As Run 14 is reasonably representative of the conditions for Section 4, it must be concluded that VESYS IIM underpredicted rut depth and overpredicted serviceability with time as indicated in Figures 84 and 88 of Appendix F.

Run 13 was made for Section 3 after the experimentation on Sections 4 and 11 was completed and inputs from the same sources used. Review of Figures 83 and 87 in Appendix F indicates that VESYS IIM seriously underestimated rut depth with time and overestimated the PSI.

Run 16 for Section 17 was also a single run as for section 3 with inputs from the same sources. Review of Figures 86 and 90 in Appendix F indicates that VESYS IIM overestimated rutting for the first 4½ years and underestimated it thereafter. The predictions for serviceability were reasonably close to the measured, but somewhat higher.

Evaluation

The shape of the predicted rut depth curve approximates that of the permanent deformation curves for the pavement materials, especially the asphaltic concrete, i.e., relatively high rates of rutting early in the pavement's life with a decreasing rate with time (or wheel loadings). This is logical for constant traffic and wheel load conditions. However, the shape of the measured rut depth curve is quite different, showing a generally steady rate for about three years and then a higher but still rather steady rate thereafter. As this increase in rate of rutting occurred approximately at the same time for all four sections, it probably represented an environmental change in the pavement materials or an increase in traffic rate that was not defined for VESYS IIM.

If a straight line through the measured data for the first three to four years is extended forward in time (representing no change in pavement materials or traffic), VESYS IIM would have underestimated rut depths for the thick A.C. pavements (Sections 3 and 4) and tended to overpredict rutting for the conventional pavements.

VESYS IIM consistently overpredicted serviceability, even during periods when it was overpredicting rut depth. The agreement was quite good for the conventional sections 11 and 17, but was very poor for the thick asphaltic layer pavements, Sections 3 and 4.

The predicted PSI for the thick A.C. sections even exceeded the initial PSI over the eight-year record. This was apparently caused by a combination of minor rutting predictions and the nature of the stochastic version of the AASHO equation for PSI.

AASHO Road Test Comparisons

The selection of test sections for the AASHO Road Test comparisons was limited to those sections for which materials samples and sufficient data were available for meaningful comparisons. Sections 266, 272, 308 and 470 were selected for study. Each of these AASHO Road Test sections had been overlaid to the current depths of the surface layers (See Table 5) and in service continually under highway traffic since 1962.

Input Variables

The input values for all variables used for these comparisons appear in Tables 5 and 6. The development of the values for materials characterizations is described in Appendices B, C and D for creep compliance, permanent deformation potential and fatigue life potential, respectively. Values for other variables either resulted from insight gained from studies described in Chapter V or were developed as described below.

The truck traffic data were sufficient to develop both numbers of trucks and mean numbers of axles per truck by year for the years 1963 through 1970. The results are plotted in Figure 15 and have been extrapolated through 1974 as shown.

There were detailed records on axle weights available that allowed development of the distribution of axle weights as shown in Figure 16. The mean axle weight was 9060 lbs. Assuming the mean wheel weight of 4530 lbs and a tire pressure of 75 psi, a RADIUS of 4.24 inches may be calculated. VCAMP was calculated to be 2644 psi by statistical means using the variance of wheel load based on the distribution in Figure 16.

The values of DURATION were calculated at the mid-depths of the surface layers as discussed subsequently in Chapter VII and based on a mean speed of 60 mph. VCDUR was calculated on a statistical basis to arrive at a reasonable value. Solutions are not sensitive to either of those variables.

Table 5 Summary of input data for VESYS IIM solutions, AASHO road test sections.

Independent Variable	Section			
	266	272	308	470
ALPHA(1)	0.87	0.89	0.88	0.88
ALPHA(2)	0.94	0.93	0.94	0.94
ALPHA(3)	0.97	1.00	0.97	0.79
AMPLITUD	75	75	75	75
BETA	0.054	0.054	0.054	0.054
COEFK1	0.30	0.30	0.30	0.30
COEFK2	0.04	0.04	0.04	0.04
CORLCOEF	1	1	1	1
CORLEXP	0.06	0.06	0.06	0.06
DELTAS		100,30,8,1, 0.1, 0.3, 0		
DURATION	0.0149	0.0156	0.0157	0.0151
GNU(1)	0.64	7.00	0.44	1.20
GNU(2)	1.64	1.91	1.60	1.75
GNU(3)	0.10	0.00	0.085	0.12
K1K2CORL	-0.867	-0.867	-0.867	-0.867
LAMBDA		2340,4000, 5220,6470, 7660,8550, 8750,9260, 9450,9700, 9800,10000		
LAYER1		See Table	6	
LAYER2		See Table	6	
LAYER3		See Table	6	
NDELTAS	7	7	7	7
NTEMPS	12	12	12	12
NTRANDOM	12	12	12	12
NTSTATIC	11	11	11	11
PSIFAIL	2.5	2.5	2.5	2.5

Table 5. Summary of input data for VESYS IIM solutions, AASHO road test sections (con't.)

Independent Variable	Section			
	266	272	308	470
QUALITYO	4.03	4.03	4.03	4.03
RADIUS	4.24	4.24	4.24	4.24
RETEMP	70	70	70	70
STDEVO	0.20	0.20	0.20	0.20
STRNCOEF		See Table	18	
STRNEXP		See Table	18	
TEMPS		See Table	18	
THICK1	11.5	12.25	12.375	11.75
THICK2	25.0	17.0	18.0	14.0
TOLERNCE	50.0	50.0	50.0	50.0
TRANDOM		1,2,3,4,5,6, 7,8,9,10,11, 12		
TUNITS	6.0	6.0	6.0	6.0
VARCOEF1	0.18	0.18	0.18	0.18
VARCOEF2	0.18	0.18	0.18	0.18
VARCOEF3	0.18	0.18	0.18	0.18
VCAMP	2644	2644	2644	2644
VCDUR	3.59×10^{-6}	3.59×10^{-6}	3.59×10^{-6}	3.59×10^{-6}

Table 6. Creep compliance arrays used for AASHO sections, D(T) ($PSI^{-1} \times 10^{-5}$)

Load Duration (sec)		0.001	0.003	0.01	0.03	0.10	0.30	1.00	3.00	10.00	30.00	100.00
Section Number	Layer											
266	1	0.045	0.063	.105	.180	.310	.480	.750	1.04	1.45	1.90	2.3
	2	6.24	6.24	6.24	6.24	6.24	6.24	6.24	6.24	6.24	6.24	6.24
	3	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
272	1	.052	0.077	0.12	0.195	0.320	0.480	0.850	1.250	1.75	2.40	3.10
	2	6.04	6.04	6.04	6.04	6.04	6.04	6.04	6.04	6.04	6.04	6.04
	3	2.35	2.35	2.35	2.35	2.35	2.35	2.35	2.35	2.35	2.35	2.35
308	1	0.042	0.057	0.097	0.150	0.256	0.410	0.620	0.870	1.23	1.56	1.72
	2	6.22	6.22	6.22	6.22	6.22	6.22	6.22	6.22	6.22	6.22	6.22
	3	3.75	3.75	3.75	3.75	3.75	3.75	3.75	3.75	3.75	3.75	3.75
470	1	0.048	0.077	0.113	0.185	0.325	0.540	0.910	1.33	2.0	2.60	3.00
	2	6.13	6.13	6.13	6.13	6.13	6.13	6.13	6.13	6.13	6.13	6.13
	3	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00

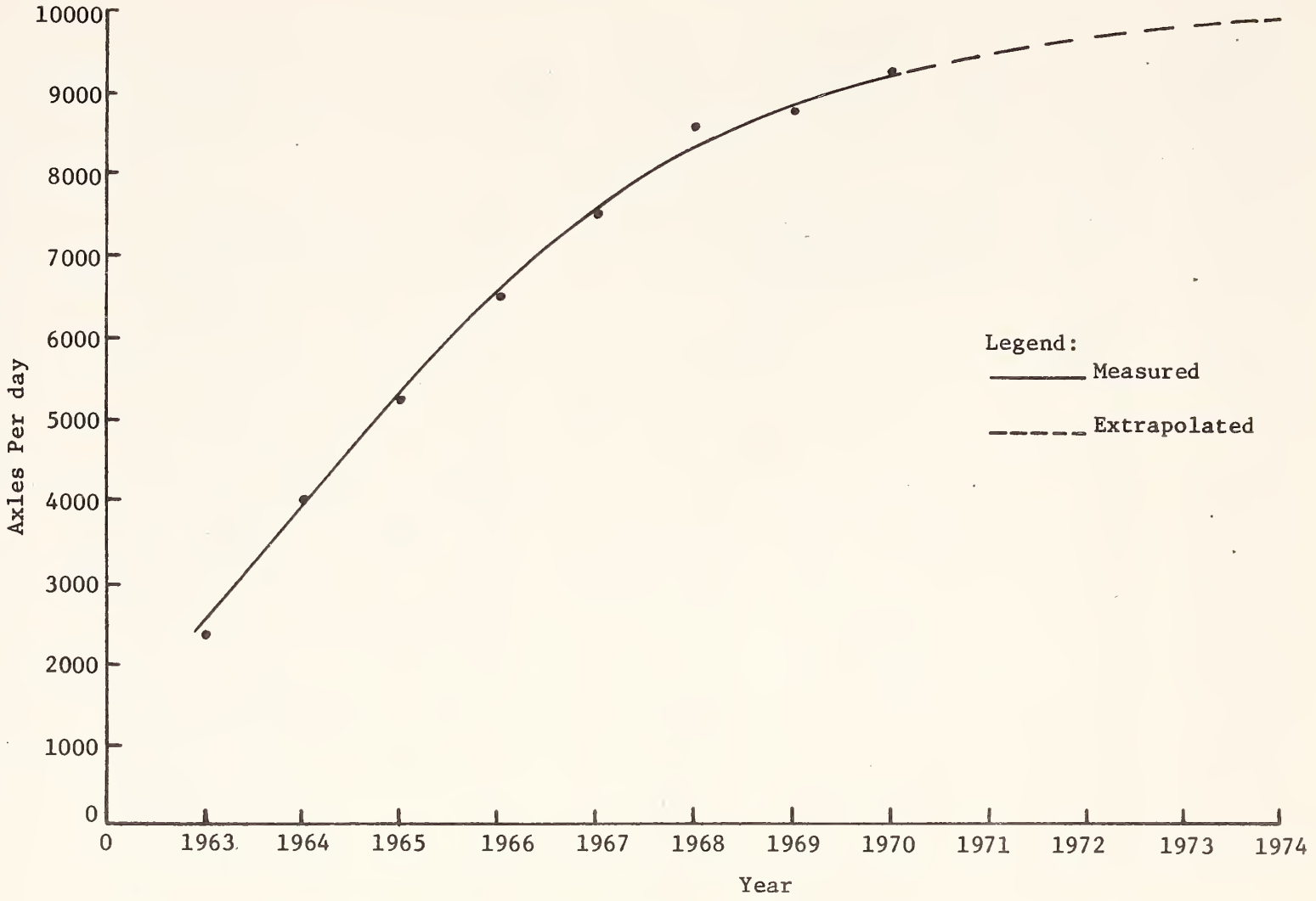

Figure 15. Average truck axles per day on former AASHO Road Test sections period of study.

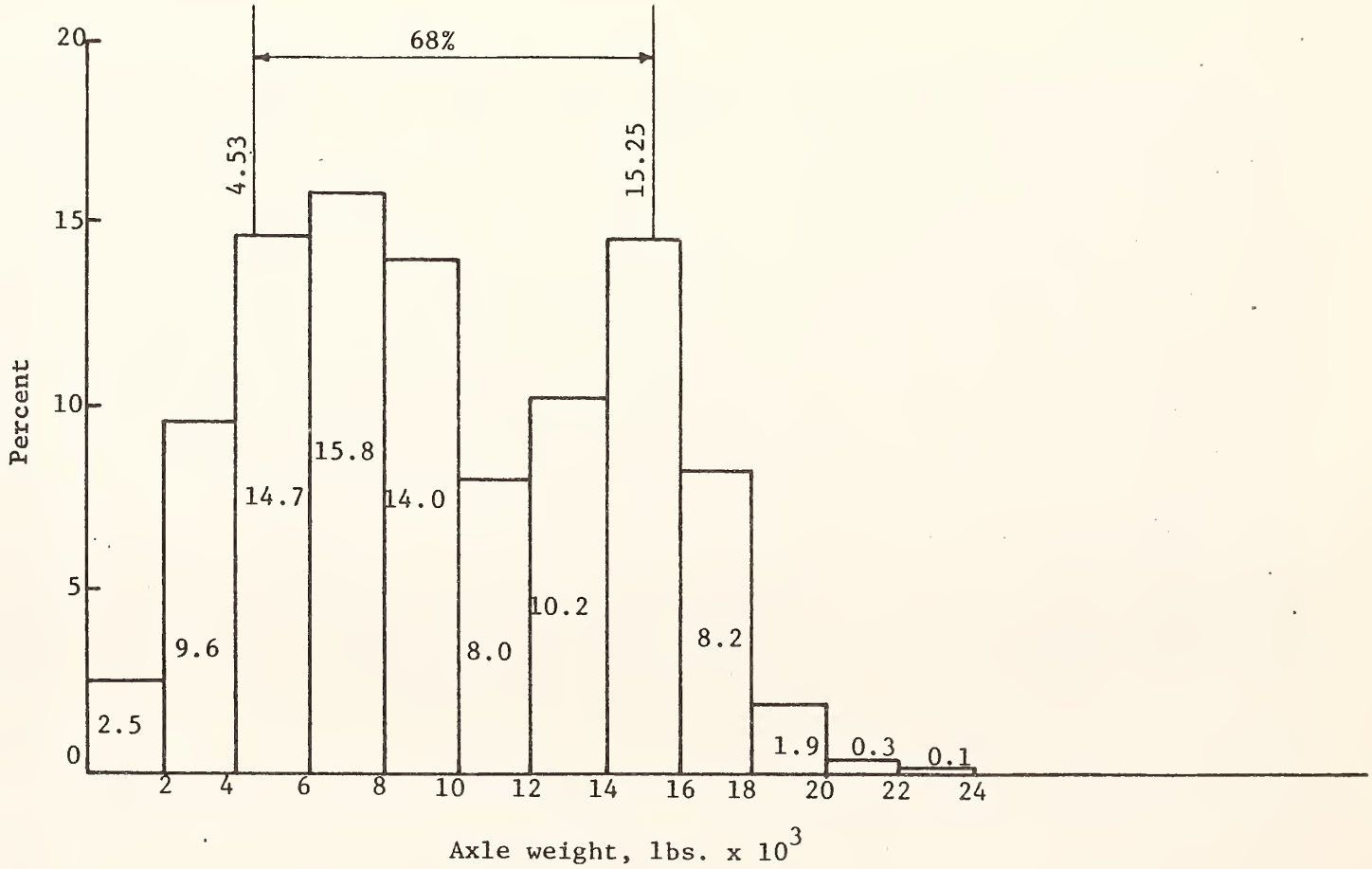

Figure 16. Axle weight distribution for AASHO Road Test sections.

The values for QUALITY0 are the initial values of Present Serviceability Index measured in 1962. The layer thicknesses, THICK1 and THICK2 are the mean actual values for the sections. The temperature array TEMPS is logically an average for Urbana, Illinois.

These input values generally represent the best available from sophisticated testing and application of knowledge gained from this research effort.

Calculated Responses

The calculated responses have been plotted in Figures 17, 18, 19 and 20 for Sections 266, 272, 308 and 470, respectively (Note that scales are not all the same). Measured values for rut depth and cracking are also plotted for comparison. As can be seen, no cracking actually occurred in any of the sections, except that there was about 3% cracking in Section 470 during the period between 1972 and 1974 measurements.

Measured values of slope variance are not included as these measurements were not made. A BPR Roughometer was used for annual measurements and the values of PSI calculated from the modified AASHO equation:

$$PSI = 10.91 - 3.9 \text{ Log } RI - 0.01\sqrt{C + P} - 1.38\ E[RD]^2 \qquad (27)$$

where:

RI is the roughness index measured with the BPR Roughometer

C is the cracking in sq. yd. per 1000 sq. yd.

P is the patching in sq. yd. per 1000 sq. yd.

E[RD] is the mean rut depth, inches.

The values of PSI calculated from measured values of RI, C, P and E[RD] were so erratic that they could not be reported with any confidence. RI is by far the dominant term in Equation (27) and it is believed that calibration from year to year was inadequate for meaningful comparisons. For instance, the calculated values of PSI in 1974 for Sections 266, 272, 308 and 470 were 4.74, 4.45, 4.15 and 4.15, respectively. All of these values are much higher than the initial values in 1962.

Evaluation

Each of these pavement sections had thick asphaltic concrete surface layers (including the overlays subsequent to the AASHO Road Test) like the Brampton "full-depth" pavement section, but also had conventional

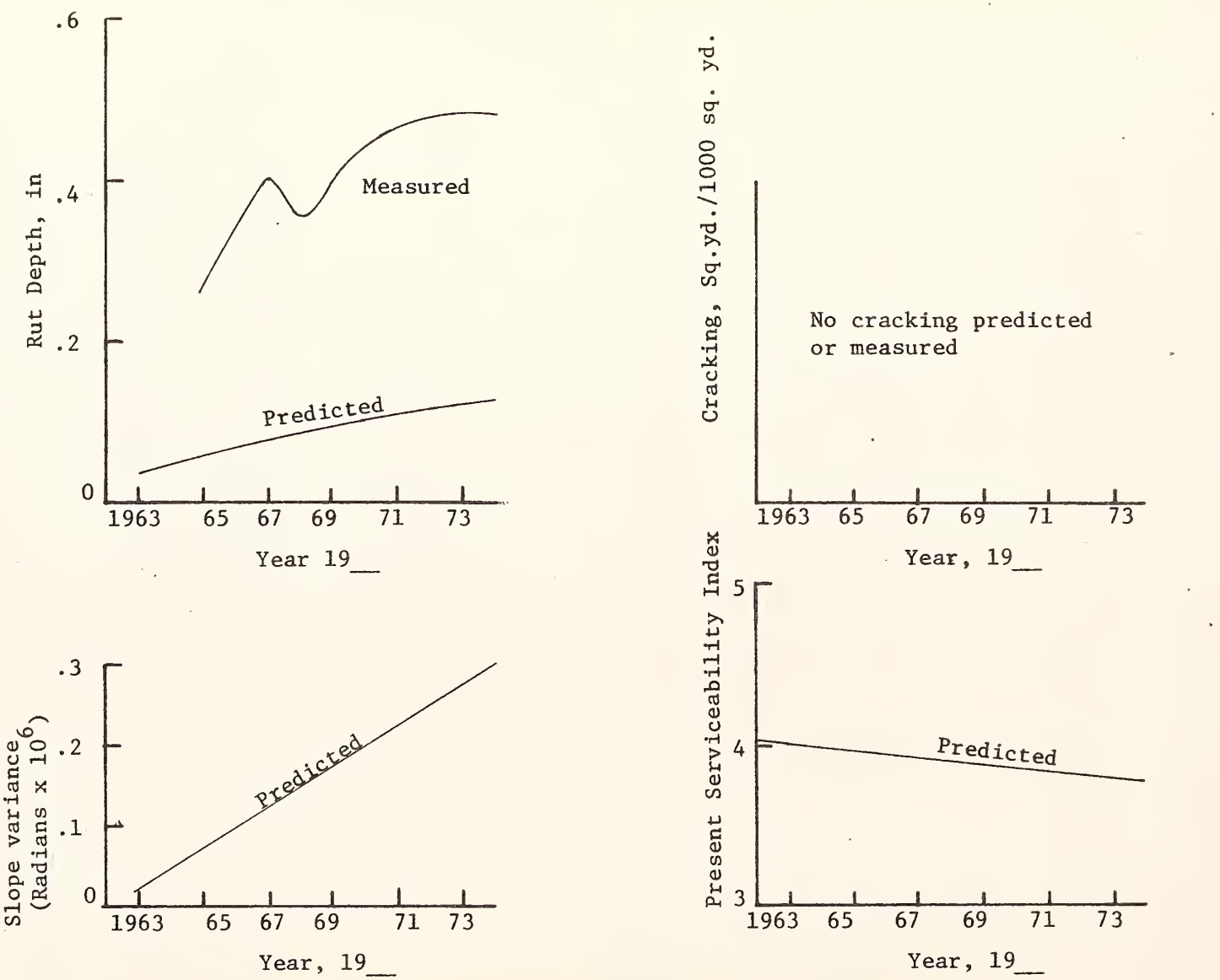

Figure 17. Comparisons of measured responses and those predicted by VESYS II (M), AASHO Road Test Section 266

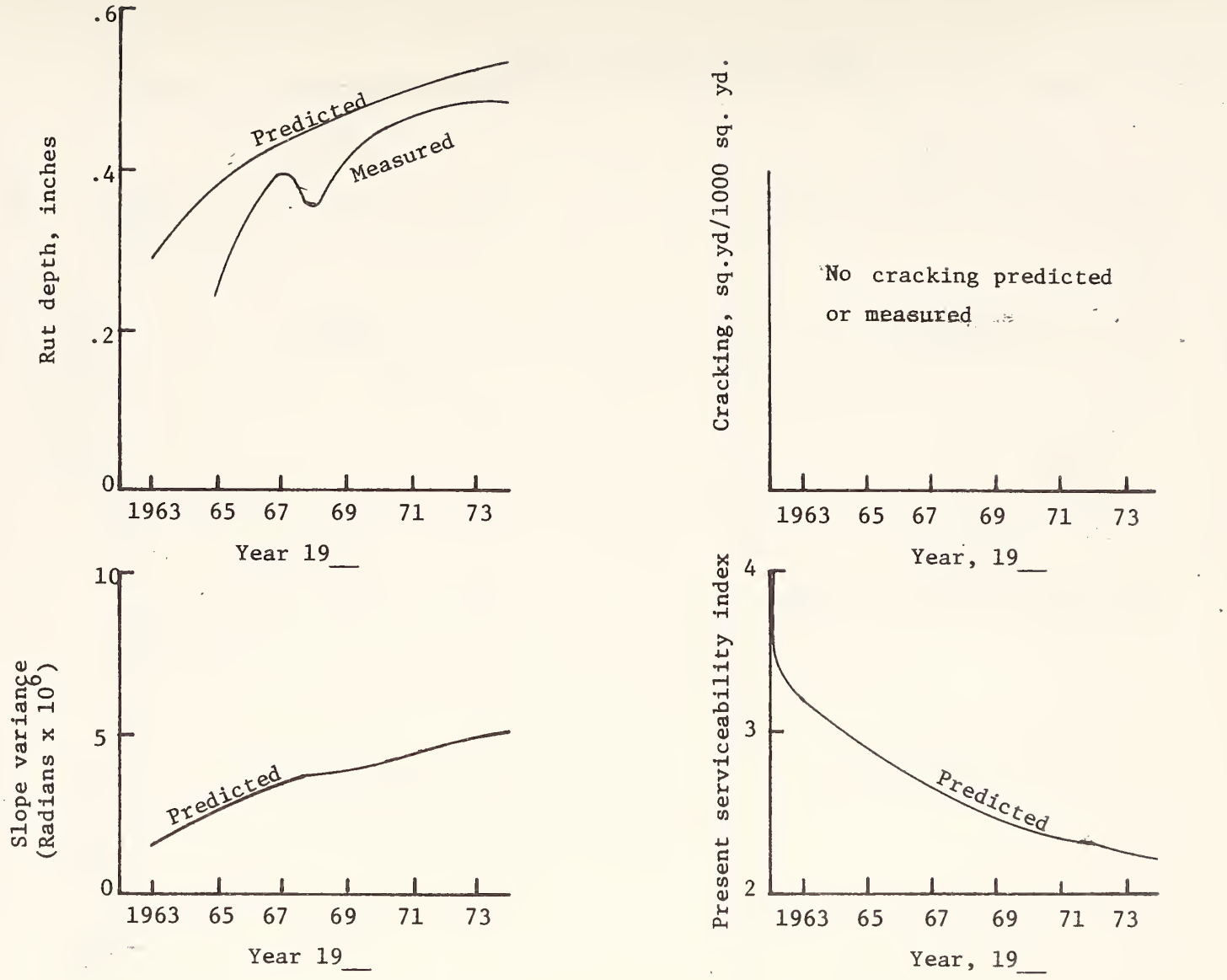

Figure 18 Comparison of measured responses and those predicted by VESYS II (M) AASHO Road Test Section 272.

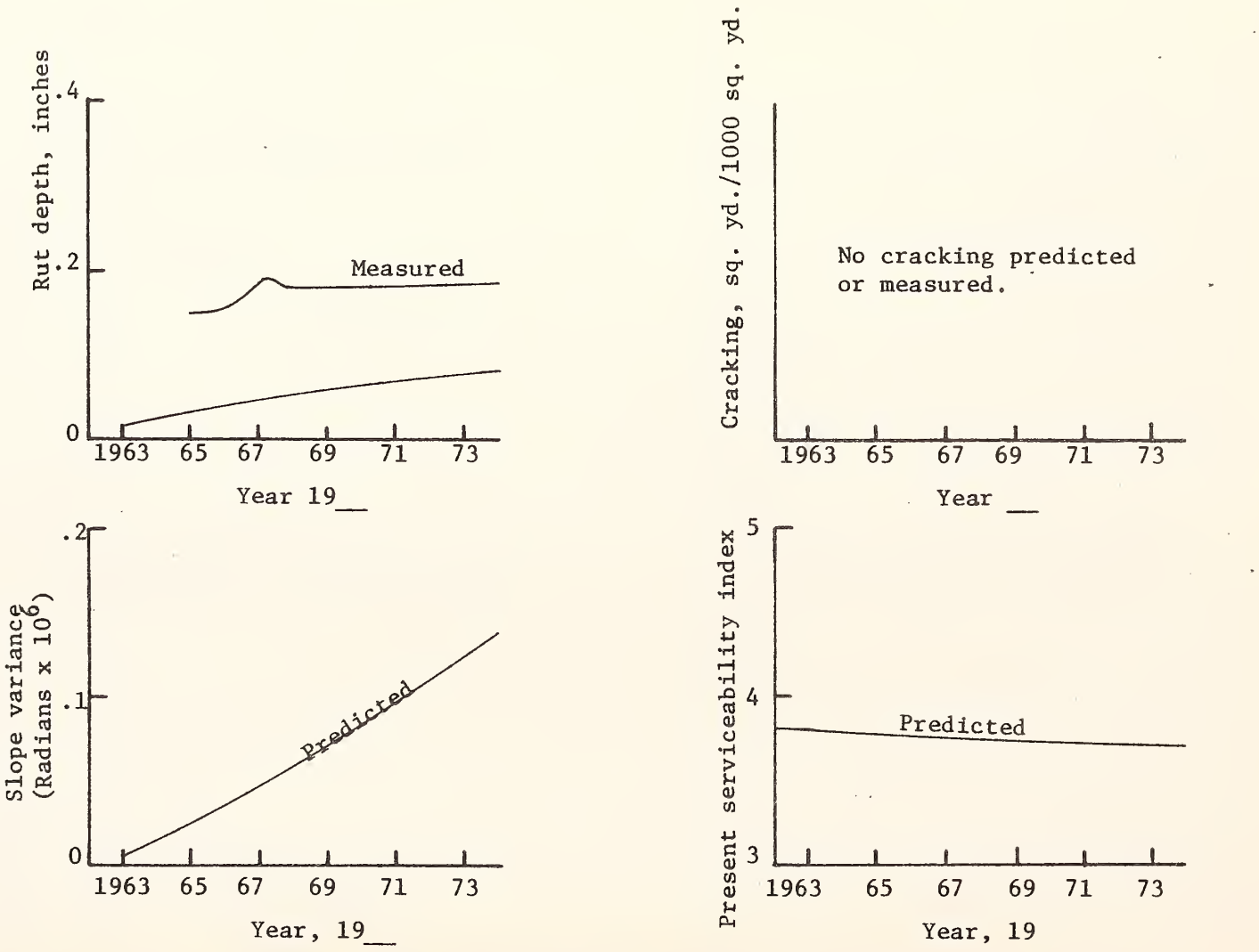

Figure 19. Comparison of measured responses and those predicted by VESYS II (M), AASHO Road Test Section 308.

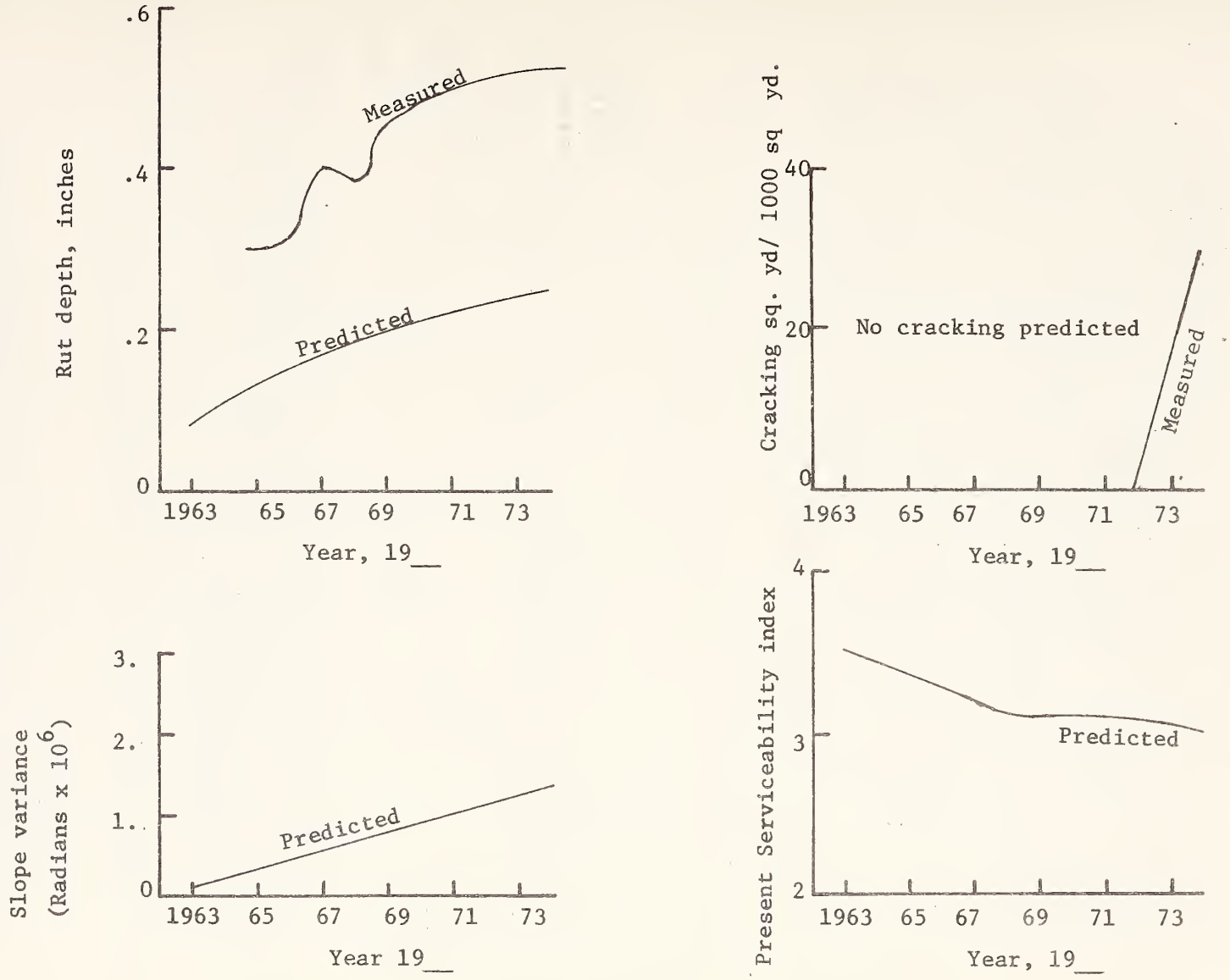

Figure 20. Comparisons of measured responses and those predicted by VESYS II (M), AASHO Road Test Section 470.

thicknesses of base and subbase. As for the Brampton sections with thick A.C. surface layers, rut depths were generally seriously underpredicted, except for Section 272.

GNU(1) for Section 272 (See Table 5) was 7.00 as compared to a range of 0.44 to 1.20 for the other sections. As GNU(1) is a fairly strong parameter (See Table 26, Volume II) and its increase results in increasing rut depth, this high value of GNU(1) is believed to have been the primary reason for its relatively higher prediction of rut depth. It cannot be attributed to layer stiffnesses as LAYER1 and LAYER2 for Section 272 were approximately as stiff as for the other sections and LAYER3 was the stiffest of the four.

As the predicted rut depths were much better for Section 272, one may speculate that GNU(1) should be higher for the other sections. This may be a reasonable speculation as Figure 57, Appendix C, indicates that high values of GNU(1) are generally consistent with low deviator stresses, which might be expected with thick A. C. layers such as these. However, decreasing ALPHA(1) would also produce the same effect and the values used represent an A. C. with little permanent deformation potential, so such speculation is inconclusive.

It is interesting to note that VESYS IIM apparently did not overestimate cracking for any of the four sections. To the contrary, it failed to predict the 3% cracking that had occurred recently in Section 470, but it did predict only 20 to 50% as many cycles to failure for this section as for the other sections. Although not shown, the calculated damage indices at the end of 12 years (1974) ranged from .0007 to .005, indicating that fatigue damage was miniscule. Review of the output indicates that the combination of relatively stiff materials and thick layers result in relatively minor radial strain predictions at the bottom of the thick asphaltic layers in the order of .000009 (at low temperature) to .000024 in/in (75^{o} F for Section 470). Predicted cycles to failure varied with temperature (both strain because of stiffness variation and fatigue life potential varied), but ranged at the 70^{o}F level from around 10^{10} for Section 470 to 1011 for Section 308.

It is interesting to note that only 3 of 50 former AASHO Road Test Sections inspected in 1974 showed any cracking and the greatest cracking observed was only 38 sq. yds per 1,000 sq. yds. This minor amount of cracking may have been caused by local weaknesses and represents only 0.16% of the pavement observed.

It appears then that the VESYS IIM idealization for predicting fatigue level may give practical results providing that:

1. The material characterization for fatigue life potential is realistic.
2. The radial strains calculated are realistic.
3. The stochastic variations are not large enough to cause

predictions for $1/N_i$ to fall in the range of divergence of the VESYS IIM functional approximation from the real function as described in Chapter VII.

Although measured values are not available for comparison, it is worthwhile to note the predicted variations in slope variance and PSI. As slope variance is the dominent term in the AASHO equation for PSI, its prediction is very important but severely limited by inability to measure the spacial auto-correlation term CORLEXP directly. As slope variance is considered also to be a direct function of rut depth and variation of rut depth and CORLEXP was considered to be the same for all sections, the predicted values for slope variance were generally small consistent with the relatively small predictions for rut depth. The exception was Section 272, for which rutting was overpredicted. Note that the result in this case was a sudden reduction in PSI during the first year when over one half of the predicted rut depth occurred and continued as rutting continued at a slower rate.

While it is quite difficult, if not impossible, to provide accurate input data for the spacial auto-correlation function and the stochastic variations of creep compliance, wheel loads, traffic and other input variables; it can be seen that the formulations in VESYS IIM do function in a rational fashion.

The service life predictions for the various sections were:

Section No.	Predicted Service Life, Years
266	39.6
272	6.8
308	105.3
470	22.7

The most interesting perhaps is the prediction for Section 272 to reach an unsatisfactory condition during 1968. As can be seen from Figure 18, this is the approximate point at which the calculated PSI decreased to the value of 2.5 specified as PSIFAIL. It is probable that the predictions for PSI are more pessimistic than justified by the difference between measured and predicted rut depth. While reliable data are lacking as to the actual PSI for Section 272, it is still in service some seven years later than 1968 and is believed to be performing better than predicted.

Ability of VESYS IIM To Predict Performance

The comparisons of VESYS IIM predictions to measured values discussed above show that performance predictions may not as yet be

made at a sufficiently high confidence level. This is not surprising, however, considering the state of the art in characterizing materials, inability of theoretical structural models in current use to deal with materials changes (such as cracks at varying stages of propagation toward the surface, seasonal moisture change in base and subgrade etc.)and other such sources of inaccuracies.

The somewhat surprising result of this study is that the calculated rut depth values approximated the measured rut depths for three of the eight sections when reasonably accurate input values were used. The odds are high against extrapolating successfully into the future on the basis of data which are not defined at a high confidence level and are subject to change with time, especially when the real responses are affected by many random events not modeled. Even this limited success by VESYS IIM suggests that the VESYS IIM predictive system holds considerable promise and may be expected to offer even better predictions as state-of-the-art improvements are made to it and to procedures for characterizing materials.

While nothing conclusive may be stated about predictions for cracking, PSI and service life, they do appear to function rationally and should also provide improved predictions as the state of the art for materials characterization and the models themselves progress.

CHAPTER VII

DEFICIENCIES IN VESYS IIM AND RECOMMENDATIONS FOR IMPROVEMENT

VESYS IIM represents an important step forward in pavement systems analysis. Moavenzadeh, his staff at MIT and the FHWA are to be commended for this innovative and promising development. However, the task of modeling the behavior of actual pavements is so complex that a considerable amount of practical work remains to be done before the computed predictions will be accurate at an acceptable confidence level. The following sections outline some of the deficiencies noticed during the course of this research and suggest research to continue improvement of VESYS IIM.

Inability to Model Decreases in Bending Stiffness Due to Cracks

Elastic layer theory assumes continuous pavement of infinite extent horizontally. Unfortunately, this condition exists only until cracks are formed on the bottom of the AC layer. Based on data from studies conducted by Shell laboratories in Holland, and on other data from other sources, most of the period of time prior to the appearance of cracks at the surface is spent in propagating cracks to the surface that have appeared much earlier at the bottom of the layer. The bending stiffness around this crack is grossly reduced during the entire propagation process. The affect of this crack is to transform the continuous pavement layer into a set of more or less discrete and much smaller continuous slabs. Radial strains are probably considerably reduced within the continuous portions of one of these small slabs as the curvature would be reduced. This would probably be accompanied by relatively high levels of shear strain at the crack in the material that was not as yet cracked through, resulting in greater displacement of the pavement surface for a wheel load on one of the small discrete slabs than for a wheel load on continuous pavement.

As VESYS IIM is dependent on elastic layer theory for prediction of strain, it is undoubtly affected by this inability to model the real system; however, this is a disability suffered by all of the newer analytical techniques based on elastic layered theory. The results would appear to be overestimation of the radial strains and underestimation of vertical displacements and vertical strains.

It is not clear as yet what must be done to render the results of elastic layer solutions more accurate when subjected to traffic during the periods after the cracking has started on the bottom of the pavement. Due to the relative unpredictability of crack pattern, it is probable that any modifications will have to be of a statistical nature.

Inability to Model Deterioration in Base and Subgrade Due to Surface Cracking or Horizontal Moisture Migration

VESYS IIM (and apparently most, if not all, other pavement idealizations) has no capability for considering deterioration of underlying materials as layer 1 cracking increases and allows excessive moisture to enter underlying layers. This could be done on the basis of reduced elastic moduli and reductions in ALPHA, increases in GNU or both. Sufficient data may exist in the literature to support a reasonable first generation model. For example, if data on changes in base and subgrade moisture contents as a function of cracking is available, this could be combined with data on resilient moduli variations with moisture content to arrive at approximate variations of elastic moduli (inverse of creep compliance) with degree of cracking.

The basic problem is that material changes in time are not considered while the predictions may cover many years of pavement service. This may not be serious as far as the base material is concerned as its permanent deformation coefficients and creep compliance were found to have only a slight effect on the calculated responses anyway. However, reductions in ALPHA(2) and increases in GNU(2) and LAYER2 outside the range considered in the sensitivity analysis might result in important effects.

The calculated responses are more sensitive to the characteristics of the subgrade, however, and serious deterioration consistent with infiltration of water through cracks, if modeled, could cause important increases in rutting and hence increased roughness and loss of serviceability and service life (as would occur in real life).

It is believed that modifications to allow for revised accumulation of predicted cracking damage and rutting would not be too difficult, but assessing the expected change in the input variables with time might not be possible at a very high confidence level. It is probable that such a modification might be worthwhile to allow study of how finite variations in material properties in time might be expected to affect the responses.

Better Approximation for the Distribution of Damage Index

In the cracking formulation (described in Chapter IV of this volume), it is assumed that damage index is normally distributed. This was clearly not the case in our sensitivity results and probably is not true of real pavements either. As is suggested in Chapter IV of Volume II, the distribution of this variable is likely to be approximately log-normal.

It is suggested that statistical tests be applied to actual fatigue data to see if the assumption of log-normality is justified. Other distributions could be attempted as well. If, as seems likely, the suggested log-normal distribution is verified, then the formulation for area cracked can easily be modified to use this distribution by changing a few cards in the program.

Modeling of Wheel Loading

From a user's point of view, the most easily misunderstood input variables are RADIUS, AMPLITUD, VCAMP and LOADING. As is explained in Appendix E, these parameters combine in a complex fashion to model the wheel loading conditions. The reason for this complexity is that RADIUS, which should ideally vary with wheel loading and tire pressure, must be fixed to a single value in order to perform the elastic layer solutions in the static load subprogram. A stochastic treatment of these calculations would appear to be prohibitively complicated. With RADIUS fixed, the only way left to model the variation of wheel loads is to vary AMPLITUD over a rather large range. This is confusing because tire pressure, which is what AMPLITUD appears to represent physically, does not exhibit a very large amount of variation. An added complication is introduced by the fact that the value of AMPLITUD is effectively multiplied by the value of LOADING, the load intensity calculated in the static load subprogram.

What is proposed is that first LOADING be removed as a parameter in a Type 1 run. Some appropriate value (probably 1.0) could be used in all such analyses. Next the rather complicated calculations required to compute AMPLITUD and VCAMP from the mean and variance of axle weights (see Appendix E) should be written into the program. This would mean that axle (or wheel) weight data could be input directly to the program and the appropriate adjustments would be made. Finally the actual mean and variance of tire pressure could be input, causing the program to compute the mean RADIUS as a stochastic function of the random variables for pressure and load.

$$r(p,\ell) = \pi^{-\frac{1}{2}}\, p^{-\frac{1}{2}}\, \ell^{\frac{1}{2}}$$

where

r is radius in inches,

p is tire pressure in psi, and

ℓ is wheel load in pounds.

Using the second-order Taylor series approximation for r about the means of p and ℓ we may write:

$$E[r] \simeq \pi^{-\frac{1}{2}} E[p]^{-\frac{1}{2}} E[\ell]^{\frac{1}{2}} + \frac{Var[\ell]}{2} \frac{\partial^2 r}{\partial \ell^2} + \frac{Var[p]}{2} \frac{\partial^2 r}{\partial p^2} + Cov\ [\ell,p] \frac{\partial^2 r}{\partial \ell \partial p}$$

where the partial derivatives are all evaluated at the means of p and ℓ.

This reduces to the following estimate of the expected value of RADIUS:

$$E[r] \simeq \pi^{-\frac{1}{2}}\ p^{-\frac{1}{2}}\ \ell^{\frac{1}{2}} \left[1 - \frac{Var[\ell]}{8E[\ell]^2} + \frac{3Var[p]}{8E[p]^2} - \frac{Cov[\ell,p]}{4E[p]E[\ell]} \right]$$

It is difficult to obtain data to accurately characterize the covariance of load and pressure, $Cov[\ell,p]$. A small positive correlation would be expected for these variables, but it seems unlikely that it would vary much from road to road. If data on this covariance could be obtained for a few roads in actual use, then the value obtained could probably be fixed in the program. The user is unlikely to have a good estimate to input for this value in a typical design problem.

Variations in Load Duration Between Layers

While the sensitivity analysis indicates that duration of wheel loading does not significantly affect the responses, the approach used is questionable and merits discussion.

The duration of the wheel loading used in selecting values of creep compliance is presently calculated on the basis of the period of time the entire footprint covers a specific point on the pavement. As the area stressed increases with depth, it appears more reasonable to utilize a duration of loading calculated for the mid depth of each layer for use in each layer. An improvement based on an approximate approach might be use of the diameter plus twice the depth to the point of interest in calculating the load duration. This amounts to spreading the load at a 45^{o} angle from the vertical.

The proposal is that the same duration now used be calculated at the surface of the pavement, but that the duration used for the first layer be revised on the basis of the radius plus the thickness of the first layer, the duration used for the second layer be based on the diameter plus twice the depth to the mid-depth of the second layer. and that the duration used for the layer three be based on the diameter plus twice the mid-depth to a level of say 18" below the surface of the subgrade. The revision of primary importance will likely be the

one to the mid-depth of Layer 1 as this will allow the relatively high viscoelastic effects of the asphalt concrete to come more into play. As the usual base materials and subgrades display very little viscoelasticity, the relatively much higher duration will actually have little or no effect.

Permanent Deformation Coefficients as Functions of Temperature and Stress

VESYS IIM is configured to carefully consider variations in creep compliance and fatigue characteristics for asphaltic concrete with temperature, but considers the permanent deformation coefficients ALPHA(1) and ALPHA(2) as invariant with temperature. It is, however, a well-known fact that little rutting occurs during the winter and most of it occurs in the summer when the pavement material is at high temperature. This has the effect of assuming that the temperature of testing for ALPHA(1) and GNU(1) is some sort of representative average for the year or that the fraction of the total strain that is permanent is invariant with temperature.

The increased material stiffness in the winter and vice-versa for the summer causes rut accumulation to vary between seasons and would apparently do so correctly if the fraction of the total strain that is permanent is invariant with temperature; i.e., that ALPHA(1) and GNU(1) are either invariant with temperature or that their combination will provide essentially identical relative permanent strains for the range of load cycles of interest. It appears that the intercept and slope of the line in the log-log plot as shown in Figure 4 may vary as long as the resultant calculated permanent strains are consistent with those using a combination of ALPHA(1,T) and GNU(1,T) not presently available.

The only data known to exist about the variation of ALPHA(1) and GNU(1) is that described in Appendix C. A rather gross multiple regression including some limited variation with temperature indicates that ALPHA(1) and GNU(1) are insensitive to temperature. While this is heartening, the data available was too limited and the R^2 for the multiple regression too low to warrant any conclusions.

There is too little known about the nature of ALPHA(1) and GNU(1) to feel comfortable with the assumptions required to consider them invariant with temperature. Additional study is required to establish the validity of these assumptions or to develop a capability for using a temperature-dependent array for ALPHA(1) and GNU(1).

It has been established that creep-compliance is more or less independent of stress, but ALPHA(1) and GNU(1) are not at all. Further, the state of stress not only affects the values of ALPHA(1) and GNU(1), but is itself extremely variable with depth in the asphaltic concrete surface layer and to a lesser degree in other layers. It would appear that, in general, the deviator stress in a surface layer will vary from zero at the surface to a quite considerable amount at the

bottom. This implies, as found in real pavements, that most of the permanent deformations would occur in the bottom portion of the pavement. In effect, VESYS IIM indirectly averages the permanent strains across the thicknesses of the layers, so this implies some representative average level of deviator stress should be applied for testing the materials for each layer.

The approach taken in evaluating permanent deformation coefficients for the AASHO Road Test Section performance predictions is described in detail in Appendix C. It basically amounted to running a series of elastic layer solutions to arrive at estimates of average stress conditions in the layers and to use those stresses in testing for ALPHA and GNU.

It is quite possible to use the elastic layer solution to predict stress levels and to select values of ALPHA and GNU to use for each layer from stress-dependent arrays internal to VESYS II (M). It is also possible to test for this data. However, this should be preceded by a factorial study to see what the effects of varying the values would be. This could likely be accomplished by using the multiple regression equations from Volume II and the crude equations for ALPHA(1) and GNU(1) as functions of deviator stress in Appendix C. Table 26 of Volume II shows immediately that this is likely not important for the base material as ALPHA(2) and GNU(2) have little effect on calculated responses over the range of values expected.

It is also fairly likely that the subgrade coefficients may not vary enough across a reasonable range of stresses to greatly affect calculated results.

It may be expected, however, that there would be some variation in calculated responses due to variations in the average values of ALPHA(1) and GNU(1) when they are arrived at in terms of variation of ALPHA(1) and GNU(1) across the depth instead of variation with stress. This is because ALPHA(1) and GNU(1) will not vary linearly with stress.

Practical Difficulties with the Slope Variance Model

Slope variance is one of the most important terms in the AASHO equation, which is used for predicting serviceability index. Unfortunately, it has the most unstable formulation. As was described in more detail in Chapter IV[1] this response is calculated from the variance of rut depth using the spacial autocorrelation coefficients CORLCOEF and CORLEXP.

$$E[SV] = \frac{2B}{C^2} \text{Var}[RD]$$

[1]This description is a simplification of the one presented in Chapter IV.

where

E[SV] is mean rut depth,

Var[RD] is the variance of rut depth,

B is CORLCOEF, and

C is CORLEXP.

There is not anything wrong with this model mathematically, but it presents several practical difficulties.

The variance of rut depth is estimated from the rutting variables and the coefficients of variation of the creep compliance data (VARCOEF1, VARCOEF2, and VARCOEF3). Since accurate values for these stochastic inputs are difficult to obtain, the variance of rut depth predicted is unlikely to be very exact.

Accurate values for CORLCOEF and CORLEXP are impossible to obtain. Perhaps some machine similar to a profilometer could be built which would be capable of gathering the data from which the autocorrelation function could be curve-fitted and values obtained for these two variables. However the enormous effort and expense which would be required does not appear to be justified. Values to be used in the sensitivity analysis were obtained by running regression on some real data to obtain values for CORLEXP, while fixing CORLCOEF to 1.0. The rationale for this procedure is explained in Appendix E in the commentaries on these two variables. While this approach at least guarantees reasonable values, it is obviously not an adequate method for determining values of such sensitive variables (note the importance of CORLEXP as reported in Chapters VI, VII, VIII and IX of Volume II).

A realistic slope variance model should take into consideration other important factors such as differential moisture change and expansive clay subgrades. Some model for predicting the expansion and shrinkage of clay subgrades such as the one in the Texas Flexible Pavement System (FPS) is recommended.

Poor Fit of Taylor Series Approximation to Fatigue Relation

Consistent overpredictions of cracking were observed with almost every set of data run. Correction of the error in the fatigue relation, which was described in Chapter IV, did not remedy this problem. Further investigation revealed that the expected number of cycles to failure (N) decreases sharply as the coefficient of variation for STRNEXP (K_2) increases. The magnitude of this decrease is larger than would seem physically reasonable. An example is given in Figure 21 where N is plotted as a function of the coefficient of variation of K_2 at

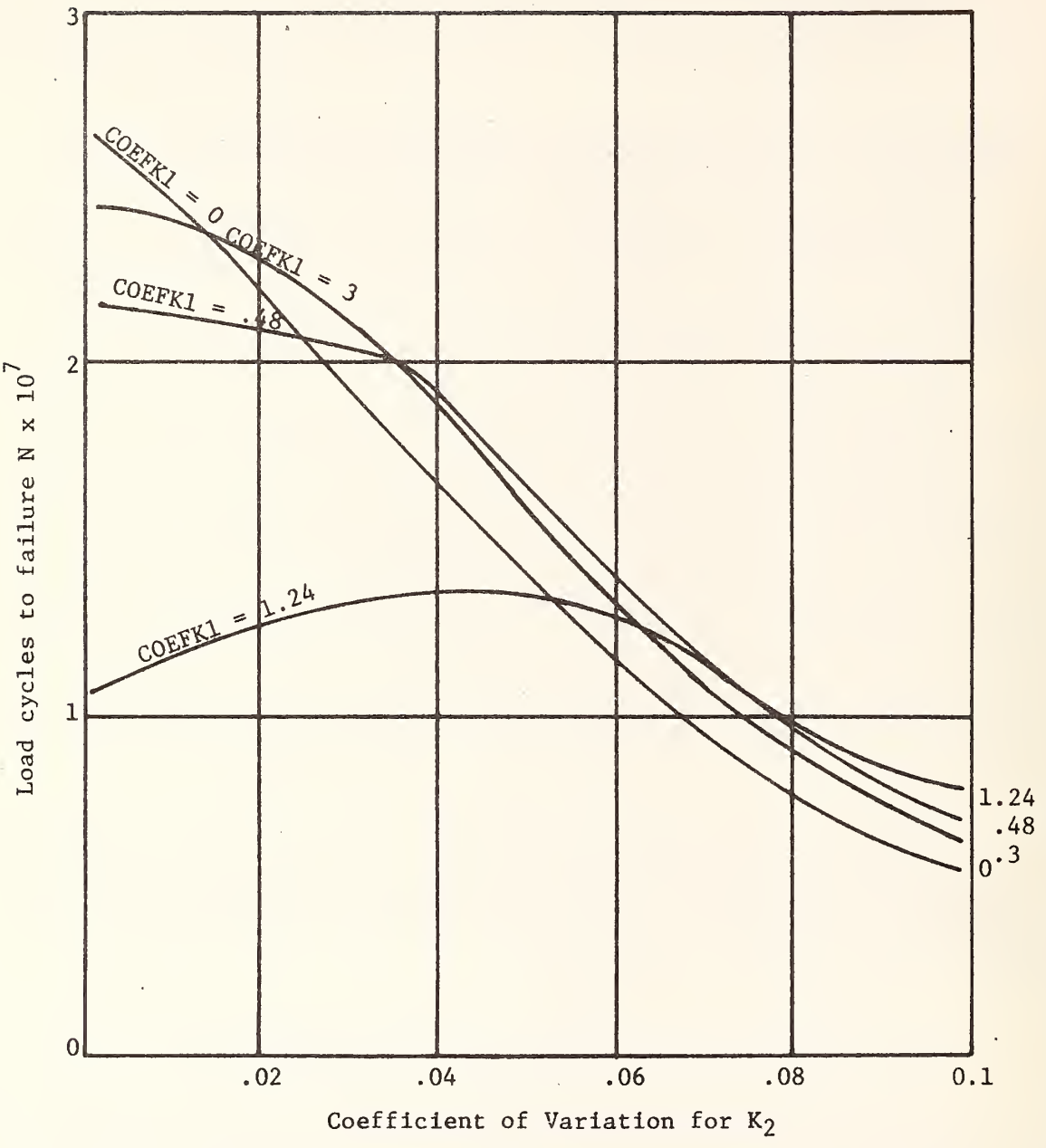

Figure 21. Fatigue life versus coefficients of variation for a material having mean $K_1 = 1.85 \times 10^{-5}$ and $K_2 = 3.04$.

several levels of COEFK1 (the coefficient of variation of STRNCOEF, K_1). The value of N at zero variation is reasonable (i.e., in agreement with test results). It is the introduction of stochastic variation which causes unrealistic predictions to be made.

In the cracking damage model, this fatigue relation is approximated by the expected value of the Taylor series expansion of the reciprocal of the number of cycles to failure. This is derived as follows:

$$\frac{1}{N} = f(K_1, K_2, \varepsilon) = \frac{\varepsilon^{K_2}}{K_1} \tag{28}$$

where

N is the number of load cycles to failure,

ε is the radial strain at the first interface

K_1 and K_2 are the values of STRNCOEF and STRNEXP.

The second-order Taylor series expansion for this relation about the means of K_1, K_2 and ε is:

$$\begin{aligned}\frac{1}{N} \simeq\ & f(E[K_1],E[K_2],E[\varepsilon]) + \frac{\partial f}{\partial K_1}(K_1 - E[K_1]) + \frac{\partial f}{\partial K_2}(K_2 - E[K_2]) \\ & + \frac{\partial f}{\partial \varepsilon}(\varepsilon - E[\varepsilon]) + \frac{\partial^2 f}{\partial K_1^2}\frac{(K_1 - E[K_1])^2}{2} + \frac{\partial^2 f}{\partial K_2^2}\frac{(K_2 - E[K_2])^2}{2} \\ & + \frac{\partial^2 f}{\partial \varepsilon^2}\frac{(\varepsilon - E[\varepsilon])^2}{2} + \frac{\partial^2 f}{\partial K_1 \partial K_2}(K_1 - E[K_1])(K_2 - E[K_2]) \\ & + \frac{\partial^2 f}{\partial K_1 \partial \varepsilon}(K_1 - E[K_1])(\varepsilon - E[\varepsilon]) + \frac{\partial^2 f}{\partial K_2 \partial \varepsilon}(K_2 - E[K_2])(\varepsilon - E[\varepsilon])\end{aligned} \tag{29}$$

where all partial derivatives are evaluated at the means of K_1, K_2 and ε ($E[K_1]$, $E[K_2]$ and $E[\varepsilon]$). If we hold K_1 and ε to their means, then Equation (29) reduces to the following quadratic in K_2:

$$\begin{aligned}\frac{1}{N} \simeq\ & f(E[K_1], E[K_2], E[\varepsilon]) + \frac{\partial f}{\partial K_2}(K_2 - E[K_2]) \\ & + \frac{\partial^2 f}{\partial K_2^2}\frac{(K_2 - E[K_2])^2}{2}\end{aligned} \tag{30}$$

Note that holding the values of K_1 and ε to any value other than their means will still produce a quadratic in K_2, but the coefficients will be more complicated. Referring back to Equation (28), we note that in the actual fatigue relation being approximated K_2 appears as an exponential term. When K_2 equals $E[K_2]$, Equation (30) gives the same result as Equation (28) but as K_2 becomes larger than its mean, the two formulations diverge rapidly. Figure 22 compares these two

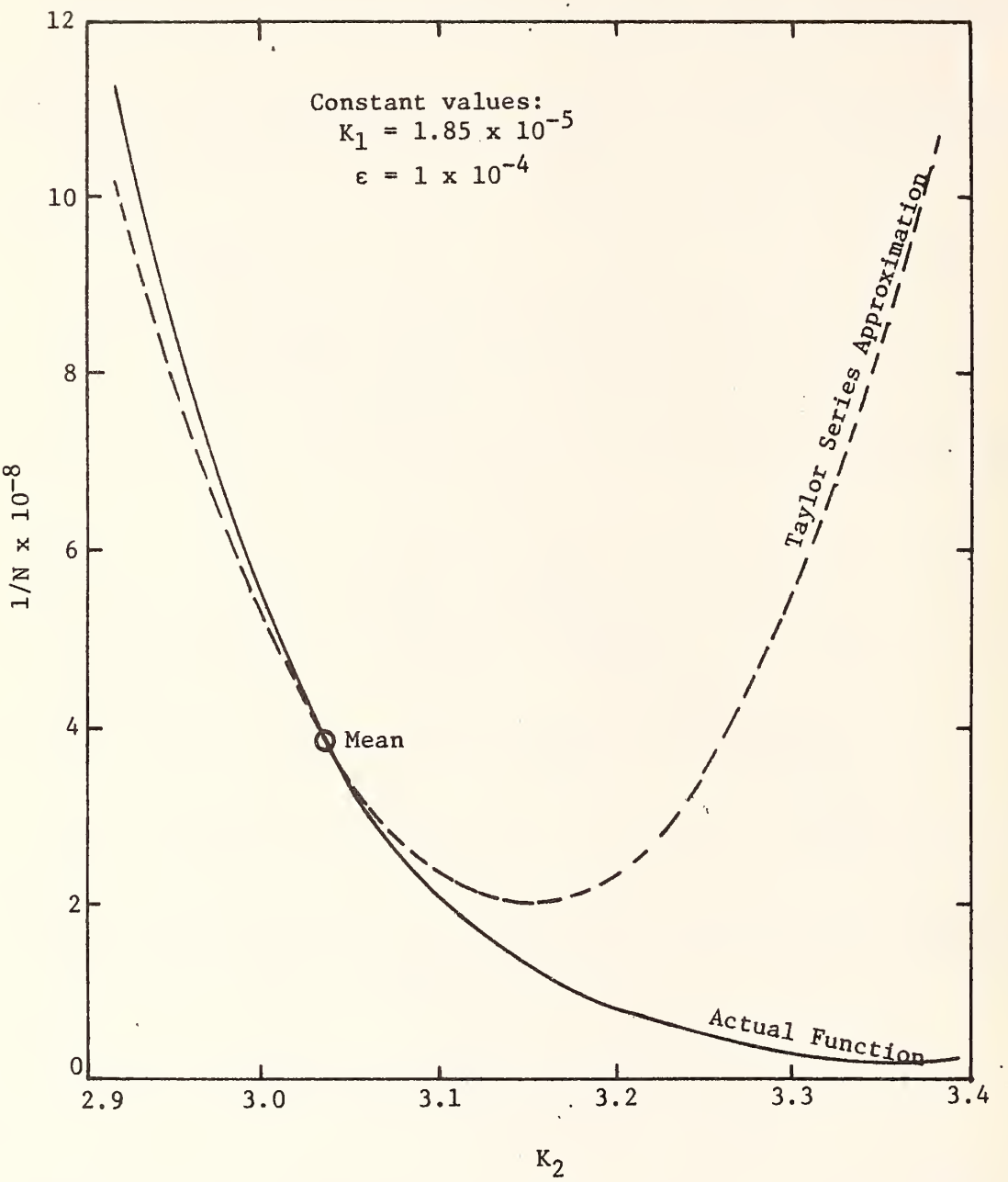

Figure 22. Graph of function 1/N and its second-order Taylor series approximation about the means of K_1, K_2, and ε.

functions with the means of the three variables set to reasonable values. The range over which K_2 is plotted might easily fall within the normal variation encountered in actual pavement materials.

Therefore, it is concluded that Equation (30), and hence Equation (29), is only an adequate approximation to Equation (28) when K_2 is held very close to its mean. When a reasonable coefficient of variation is used for K_2, we find that the Taylor series approximation seriously overpredicts 1/N (hence underpredicting cycles-to-failure) over a significant portion of the distribution of K_2. Hence it is not surprising that the expected value of Equation (29) produces excessive cracking unless COEFK2 is set to a very small value.

Realistic cracking predictions cannot be expected until a better approximation to Equation (28) is found for use in the stochastic formulation of this fatigue relation.

Incomplete Testing of the Computer Program

Although many errors have been identified and removed, occasional problems still arise. This is because the program has been extensively modified over the past year and a half, yet a really complete test of all parts of the code has not been performed. The sensitivity analysis does not qualify as such a test because it excercises only one pathway through the code. Before this program is distributed for widespread use, a comprehensive set of test data should be built up that checks out every pathway through the code. Such data might be run as a factorial of the program's "control variables," exercising all of the types of analysis provided. When future modifications are made, this test data should be rerun to verify that seemingly unrelated sections have not been affected by the changes. This same set of data must be provided whenever the program is sent to a new installation. The programmers performing the implementation will then have a good chance to discover any incompatabilities before production use begins. The consequences of hidden "bugs" appearing during production use of a program to design expensive highway facilities could easily be serious.

Difficulty in Assigning Realistic Values to Unusual Input Parameters

VESYS IIM requires material characterizations that demand rather sophisticated testing capabilities and very expensive test programs. It will likely not be feasible to include such a test regime as a regular portion of a design procedure.

There are several approachs available to deal with this problem. One such is to use simpler tests such as static indirect tension tests correlated to other properties. While this may be feasible in some cases, it requires a reasonable level of correlation between the

properties that may not always exist.

Another approach might be to correlate the results of short-term dynamic tests to much longer tests so that the results of a test one work day in duration may be extrapolated to the anticipated number of design wheel loadings.

A much more direct approach, but a somewhat expensive one, is to fund a comprehensive test program to thoroughly test materials with suitable ranges of properties (for asphalts this would include percent asphalt, relative stiffness of asphalt cement, aggregate types and gradation, percent air voids, density and probably others). The intent would be to arrive at multiple regression equations that would give reasonable accurate predictions of the materials properties used by VESYS IIM so that users without sophisticated testing capabilities could use VESYS IIM with sufficient confidence. It is probable that suitable statistical experiment design could restrict the factorial of test parameters to a fractional factorial that would not exceed reasonable funding requirements.

It is probable that some device such as the Shell Laboratories Wheel Tracking Test device will be required to give rational values of the fatigue coefficients K_1 and K_2.

The stochastic variability of most parameters is difficult to define, but there have been efforts to quantify these variables in recent years. Reference 12 summarizes a considerable amount of this research and this report should be useful in establishing some values for approximate use.

It will probably be difficult to gain approval of the large test program suggested above, but it is the authors' belief that it would prove more economical in the long run than funding a number of smaller, less coordinated studies that may not truly satisfy the requirement.

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

A very considerable effort has been expended toward gaining a thorough understanding of VESYS IIM, developing realistic values for and evaluating many independent variables, comparing its performance predictive capabilities to real pavement performance, assessing its shortcomings and correcting several of them. An overall assessment of VESYS IIM may be made on the basis of this experience, but careful objectivity is required to insure that the assessment is in terms of reasonable expectations.

This chapter is aimed at a discussion of VESYS IIM as a state-of-the-art flexible pavement analysis model, its limitations, its potential for continued development and what needs to be done next.

VESYS IIM As A State-Of-The-Art Flexible Pavement Analysis Model

A number of capable persons have contributed to the development of this very complex computer program and the idealizations of engineering behavior that it includes. Both past technology and new concepts have been incorporated to build toward a rational overall model for pavement system behavior. Some questions that have arisen on the sensitivity analysis are both posed and answered below:

1. Does VESYS IIM calculate reasonable predictions for the pavement responses important to the AASHO serviceability equation?

   Answer: It sometimes does, but may not as yet be relied on to do so at a sufficient confidence level in all cases.

2. What effect does the viscoelastic treatment have on predicted responses (rutting, fatigue life, slope variance, PSI and service life)?

   Answer: Apparently not much effect for moving vehicles as the load duration is too short for appreciable creep. However, the use of master creep compliance curves for the A. C. does allow consideration of longer load durations and definition of layer stiffness with temperature.

3. Are the techniques used for rut depth predictions rational?

Answer: They appear to be rational, but require materials characterizations that are new and sophisticated.

4. Are the techniques for the very important prediction of slope variance rational?

   Answer: The technique used is statistically well posed, but the input variables are difficult to quantify and both initial slope variance built in during construction and that resulting from other causes besides rutting are ignored. The roughness caused by swelling clays in some areas is an important example.

5. Does VESYS IIM accurately predict serviceability and service life?

   Answer: It does in some cases make reasonable predictions, but PSI is heavily dependent on slope variance predictions and service life on PSI. Thus, improvements in the slope variance model are needed to improve predictions of PSI and service life.

The questions and their answers presented above are both positive and negative as to VESYS IIM's capabilities. We believe it necessary at this point to consider carefully what should be expected. As a comparison, do the semi-empirical, semi-theoretical design procedures in common use even attempt to make such predictions? With the exception of a few recent systems not really in common use either (such as the Texas FPS or SAMP systems), they neither attempt detailed predictions, in time or expect to deal directly with the distress manifestations such as rutting and cracking. Most methods are satisfied when designs are arrived at that have performed reasonably under similar conditions in the past. It appears then that it is enough to expect of VESYS IIM that it provide a rational predictive framework with sufficient promise of future relative success to warrant its continued development.

One of the results of this research effort has been greatly improved understanding of VESYS IIM and its relative sensitivity to its many input variables. A second result has been a better understanding of and some improvements in techniques for testing to obtain the necessary materials characterizations. On the basis of this insight, we have concluded that the idealizations in VESYS IIM generally represent the state-of-the-art and some represent distinct advancement of the state-of-the-art. While VESYS IIM is obviously not the finished product, it is a very good candidate for continued development toward the goal of a comprehensive rational pavement system model.

Limitations of VESYS IIM

The limitations of VESYS IIM have been previously discussed, especially in Chapter VII. Their seriousness is related to whatever priorities are attached to the various calculated responses. For instance, if the prediction of serviceability and service life are considered primary, then the shortcomings of slope variance predictions are important and those for cracking predictions unimportant (cracking can only change a serviceability prediction by 0.3 if totally cracked). If fatigue failure is deemed important due say to subsequent deteriorations expected due to the cracks, then a better stochastic approximations for the fatigue equation is a priority item. The point here is that the evaluation of these limitations and establishment of priorities for their correction is dependent on the relative importance attached to the separate predicted responses.

The limitation of VESYS IIM's predictive capabilities fall generally into two categories. Those that are inherent in its formulation and those related to the user's capability to accurately define the input variables. None of the specific deficiencies or categories appear to be beyond reasonable correction, but the "degree of correction" will be dependent on the effort that can be expended.

It is important to keep in mind that most of the deficiencies noted in Chapter VII are not unique to VESYS IIM, but represent limitations in the state-of-the-art that apply generally to any structural model.

Recommendations for Continued Development of VESYS IIM

For the purpose of these recommendations, it will be assumed that more accurate predictions of cracking and fatigue damage are desired, but that improvements to the rutting and slope variance models are considered more important. On this basis, the following recommendations are offered in general priority order:

1. Improve the slope variance model by:

    a. Development of a means for estimating initial slope variance, perhaps from an approximate calculation using the AASHO equation and the initial PSI.

    b. Include the effects of swelling clay subgrades, probably by adaption of the procedures developed for the Texas Flexible Pavement System (FPS).

    c. Conduct a detailed study of the statistical approach used to predict slope variance to see if a more direct

formulation may be developed that might eliminate the need for the materials variable CORLEXP in the spatial auto-correlation function. This is desirable because CORLEXP is very abstract and difficult to quantify. If a better formulation is not found, special studies will be needed to better define and quantify CORLEXP.

2. Improve the capability for rut depth predictions by:

   a. Continued study of the permanent deformation coefficients ALPHA and GNU to allow more confident quantification. This may include further development of testing techniques and fractional factorial testing over a range of properties to develop multiple regression relationships. These could be used by those without the necessary sophisticated testing capability.

   b. Development of a means for directly considering the apparent stress-dependency of ALPHA and GNU. This might be done approximately through correlations with say relative layer thicknesses, wheel loadings and creep compliance, but could also be done through stress calculations within the elastic layer analysis and input arrays of ALPHA and GNU versus stress.

   c. Further study and development of creep compliance testing to insure that the input arrays are realistic. Conditioning of the specimens prior to testing is considered to be paramount.

3. Further development of the stochastic fatigue equation to improve its approximation to the real function $1/N_i$ at the higher levels of stochastic variation.

4. Continued development of fatigue relationships for materials under stress conditions simulative of real pavements. Also continued development and study of their variations with temperature.

5. Study of VESYS IIM predictions for pavement structures designed by procedures in common use. As the procedures in use have an empirical background, very poor performance predictions for designs known to generally perform reasonably should lead to identification of possible weaknesses in VESYS IIM. The quantified predictions might also lend insight as to possible weaknesses or overdesigns by the design procedure. Such comparisons are currently being implemented in two states by the FHWA.

6. Conduct a study to see if variability of material properties with time, traffic, environment or other parameters may be developed to represent deterioration of pavement materials with time. This variability could be combined with the stochastic variables already

used to represent "between-project" and "in-project" variations, but might have to be time or traffic dependent and introduce the need for more elastic layer solutions and Dirichlet curve fits.

7. Other improvements to correct deficiencies defined in Chapter VII, but not included above. These are described in Chapter VII under the subsections listed below:

   a. Better approximation for the distribution of damage index.

   b. Modeling of wheel loading.

   c. Incomplete Testing.

Summary Conclusions

VESYS IIM has been thoroughly studied along with its independent variables; realistic values for the inputs have been developed and an extensive sensitivity analysis conducted (see Volume 2 of this report), VESYS IIM's pavement performance predictive capability has been tested against measured performance for real pavements, its deficiencies have been defined and several corrected; and studies have been recommended for further improvement. These accomplishments as described in this report (Volumes 1 and 2) are believed to satisfy the goals established for FHWA Contract No. DOT-FH-11-8258.

Based on the experience gained from this effort, general assessments of the VESYS IIM Pavement Analysis and Design System have also been proffered. In summary, VESYS IIM is not as yet the finished product, but represents a significant advancement of the state-of-the-art for analysis of pavement systems.

The development of a rational analytical capability for flexible pavements is an ambitious long-term goal, but the return on the investment is potentially large. Substantial progress has been achieved in the past decade by numerous organizations across the country. This effort must be continued and the best from the various pavement analyses models should be brought together and integrated. VESYS IIM may offer the best structural framework for this integration; the FPS and SAMP systems offer optimization of designs, swelling clay models and other developments; and other systems may be expected to offer significant improvements. It would be a great mistake for the profession to expect too much too soon and to abandon this promising long-term development effort.

APPENDIX A

PAVEMENT LIFE

Notation

Reliability theory is concerned with predicting time-to-failure. This is a random variable whose probability density function is, in general, unknown. The following symbols will be used in this appendix:

L = time-to-failure.

$f(t)$ = probability density function (p.d.f.) of time-to-failure.

$F(t) = \int_0^t f(x)dx$ = cumulative distribution function (c.d.f.), of time-to-failure. This represents the probability that a section of pavement will fail at or before time t.

$R(t) = 1 - F(t)$ = Reliability of pavement. This is the probability that a section still functions satisfactorily at time t.

S_i = present serviceability index of pavement at time t_i.

S_{fail} = failure level of serviceability (PSIFAIL).

$E[x]$ = mean (expected value) of a random variable x.

$Var[x]$ = variance of x.

Expected Life Model

The expected value of time-to-failure can be written:

$$E[L] = \int_0^\infty t f(t)\, dt \tag{31}$$

Since $R(t) = 1 - \int_0^t f(x)\, dx$ we have $\frac{dR(t)}{dt} = -f(t)$

Therefore

$$E[L] = \int_0^\infty -t \frac{dR(t)}{dt}\, dt$$

Integrating by parts we get

$$E[L] = \left[- tR(t)\right]_0^{\infty} - \int_0^{\infty} -R(t)\, dt$$

$$= \left(\lim_{t\to\infty} -tR(t) - 0\right) + \int_0^{\infty} R(t)\, dt$$

The pavement will not last forever, there must be some finite upper bound on service life, t_{max}, such that $R(t_{max}) = 0$. For all $t > t_{max}$ we have $-tR(t) = 0$. Hence

$$\lim_{t\to\infty} -tR(t) = 0$$

From this we conclude that:

$$E[L] = \int_o^{\infty} R(t)\, dt \qquad (32)$$

Estimates of reliability as a function of time are obtained from the mean and variance of serviceability which are calculated at each point in the TRANDOM array specified by the user. Serviceability is assumed to be a random variable with normal distribution. Figure 23 depicts this process graphically.

Let S_i be the serviceability at some time $t_i \in$ TRANDOM, and assume $S_i \sim N(E[S]Var[S_i])$. Then the probability density function of S_i is:

$$f_N(S_i) = (2\pi\ Var[S_i])^{-\frac{1}{2}}\ e^{-\left[\frac{(S_i - E[S_i])^2}{2\ Var[S_i]}\right]}$$

and $$R(t_i) = 1 - \int_{-\infty}^{S_{fail}} f_N(S_i)\, dS_i$$

Computational Methods

Estimates of R(t) are obtained for each time t_i specified by the user for repeated load calculations. Expected life is then approximated by the integral over this time period using the trapezoid rule:

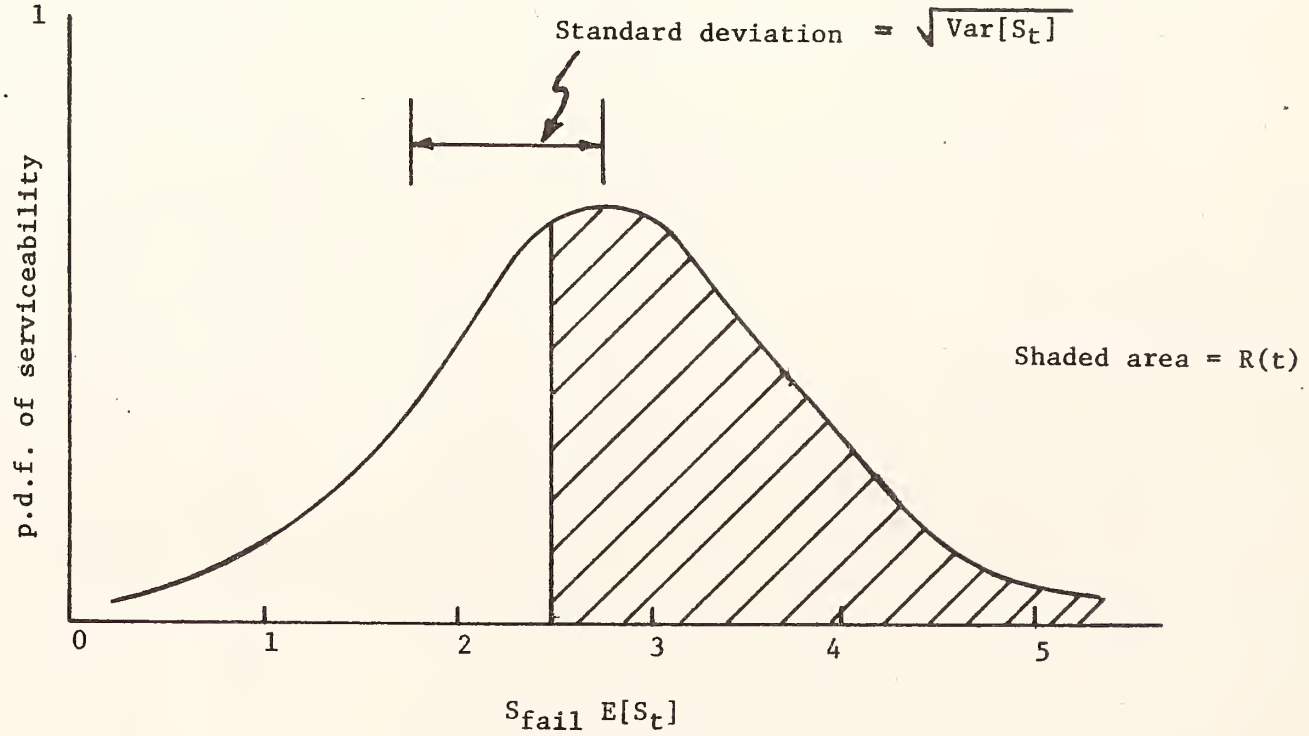

Figure 23. Method of estimating reliability from mean and variance of serviceability assuming normal probability density function (p.d.f.).

$$E[L] \simeq \sum_{i=1}^{n} (t_i - t_{i-1}) \left(\frac{R(t_i) + R(t_{i-1})}{2} \right)$$

The most important error term in this approximation is

$$\varepsilon_{tail} = \int_{t_n}^{\infty} R(t)\, dt \qquad (33)$$

There are other computational errors involved in the use of the trapezoid rule, but they can be expected to be relatively small compared to ε_{tail}. We can get a feel for the magnitude of this error by using a very large value for t_n (say 160 years) and comparing the resulting expected lifetime to that obtained using the same input data but a shorter analysis period (such as 12 years).

Such comparisons have been made. For example, one set of data which was taken from one of the AASHO sections yielded an expected life prediction of 3.2 years with t_n = 12 years, but raising t_n to 160 years resulted in an expected life of 14.8 years.

Which of these divergent estimates is correct? From a numerical point of view, the second value more nearly approximates the right side of equation (32) because $R(t_n)$ is equal to .06, and consequently ε_{tail} is minimized, as can be seen from equation (33). Yet from an engineering perspective this estimate is a very poor measure of pavement life because the reliability at 14.8 years was about .14, indicating that there is only a 14% probability of a given pavement actually lasting that long. For this reason it was decided that a better measure of service life than E[L] would be required.

New Service Life Model

The formulation which has been implemented to replace expected life as a measure of pavement performance is called service life. It is based directly on the concept of reliability. A user of the program specifies the minimum acceptable reliability either explicitly or by default and service life is defined to be the time at which this reliability is reached. Stricter tolerances can be applied for facilities in which failure would be particularly undesirable. If no value is input for this variable, then 50% reliability is considered the minimum, which means that the pavement will be declared to have failed when the expected value of serviceability reaches S_{fail} (the specified failure level, PSIFAIL).

Service life is computed in the program by linear interpolation in the reliability array. The only difficulty arises when the minimum reliability is not reached by time t_n. In this case it is necessary

to extrapolate, a risky business at best. Consequently a message is printed to warn the user that extrapolation has been performed and explaining that a more reliable estimate may be obtained by increasing t_n out past the projected life.

The extrapolation technique which was employed assumes that after some initial settling, time-to-failure approximates a Weibull distribution. The probability density function for this distribution is:

$$f(t) = \alpha\beta t^{\beta-1} e^{-\alpha t^{\beta}} \tag{34}$$

where $\alpha>0$ and $B>0$ are parameters which are fitted to the particular distribution being described. This is a very flexible function which is useful for describing the reliability theory of many different types of systems (note that for B=1 this reduces to an exponential distribution). The reliability function for the Weibull distribution is derived as follows:

$$\begin{aligned} R(t) &= 1 - F(t) \\ &= 1 - \int_0^t \alpha\beta x^{\beta-1} \; e^{-\alpha x^{\beta}} \; dx \end{aligned}$$

Substituting $u = \alpha x^{\beta}$ we get:

$$\begin{aligned} R(t) &= 1 - \int_0^{-\alpha t^{\beta}} - e^{u} \, du \\ &= 1 + \left[e^{-\alpha t^{\beta}} - e^{0} \right] \\ &= e^{-\alpha t^{\beta}} \end{aligned} \tag{35}$$

Taking the natural log of equation (35) we get:

$$-\ln R(t) = \alpha t^{\beta} \tag{36}$$

which is a straight line on a log-log graph.

Figure 24 shows a typical reliability curve computed by VESYS plotted using this transformation. Note that after the initial curvature, the graph flattens out fairly well to a Weibull distribution past about 1.5 years, although there is always some slight curvature. Every computed reliability prediction investigated exhibited similar behavior. Extrapolation on the basis of equation (36), drawn with dashes in Fig. 24, will tend to slightly overpredict the value of $-\ln R(t)$ thereby providing a somewhat conservative estimate of reliability (and hence service life). This extrapolation is based on the

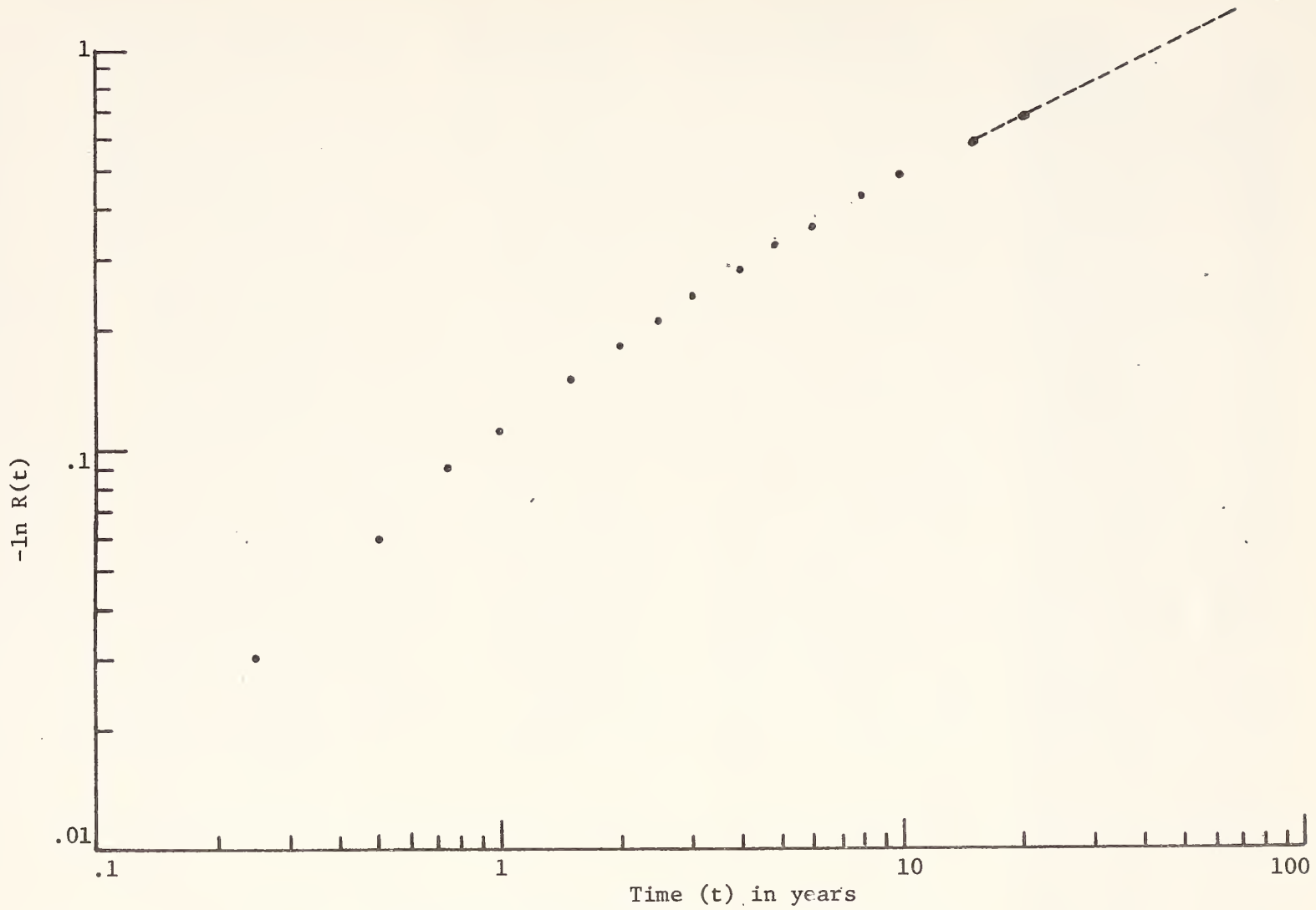

Figure 24. Method used for extrapolating reliability.

last two points only. If the whole curve had been fitted to a line, the initial curvature (during the first year) would have had an undue influence on the estimate.

It is not at all unusual in reliability theory for a system to exhibit different failure-rate behavior in different parts of its lifetime. In the real world, a new facility tends to have a failure rate which is characteristic of construction errors, while later its failures will be due to wearing out. For this reason it makes sense to ignore the early behavior of the pavement when attempting to predict reliability after many years.

APPENDIX B

DEVELOPMENT OF REASONABLE RELATIONSHIPS FOR CREEP COMPLIANCE

The approach taken was to accumulate all the creep compliance values for asphaltic concrete base materials and subgrade materials from literature research and other sources and evaluate them statistically. It was necessary to combine data at different temperatures for asphaltic concrete to arrive at master creep compliance curves and the amount of such data was limited.

Creep compliance tests are basically static load tests over relatively long periods of time compared to the fraction-of-a-second wheel loads from moving vehicles. The best available test for simulating these rapid dynamic loadings is the dynamic test that is used to gain the "dynamic modulus" for materials. Hundreds or thousands of loads may be applied for 0.1 second duration or even less to simulate repetitive wheel loading. If creep-compliance test results are to reasonably represent the stiffness of pavement materials under service conditions, there should be reasonable agreements between the dynamic modulus and the inverse of creep compliance for a load duration of around 0.1 second or less.

The predicted strains used for cracking and rutting predictions in VESYS IIM appeared to be much greater than usually calculated by elastic theory, so the relative values of creep compliance were checked between asphaltic concrete cores from the AASHO test pavements and it was found that the static creep compliance test gave values that were around ½ or less of the dynamic modulus obtained from the same sample.

It was reasoned first that creep compliance values over the short load period during the dynamic test might be the appropriate values to use as they were obviously consistent with the dynamic values, but this only gave values for 0.1 second load duration or less. It was then reasoned that the dynamic conditioning was the important factor and that a static creep compliance test immediately following the dynamic test would yield consistent results also. Testing indicated this to be the case and the results of this testing were used in lieu of static creep compliance values in the literature. The results from the literature are reported because they had already been accumulated. The development of "dynamic creep compliance" values are also discussed subsequently.

Static Creep-Compliance Tests

This section summarizes the data collected for static creep-compliance tests. These data were originally to serve as input for

the VESYS IIM program sensitivity analysis; however, for the reasons discussed above, their use is no longer considered feasible. The present task is to summarize and catalog sources of static creep-compliance test data already accumulated for availability should future need for the data arise.

Creep compliance (strain divided by stress or the inverse of the elastic modulus) is calculated from measured strains after certain durations of load in a constant-stress test. Creep compliance is usually plotted versus corresponding load durations in time and has the units psi^{-1}.

Creep compliance curves are only useful for viscous materials. As granular base and subbase materials generally respond to a load immediately with all the strain that will occur while the load is applied (i.e., elastic response), the creep compliance curve will be level and creep compliance testing is unjustified. It will be adequate to use the inverse of the resilient modulus for the creep compliance of granular base and subbase materials.

Clay subgrade materials do creep under a continuous load, but the responses are essentially elastic under short-term dynamic loads that simulate wheel-load durations and the low deviator stresses typical for subgrades (i.e., there is no appreciable increase in strain under load). In the past, static creep compliance tests to 1000 seconds at relatively high stress conditions have been used to characterize subgrade material for VESYS IIM. The resulting strains are very stress-dependent and are also heavily dependent on stress history. If it is desired to use such a test, it should be run at a stress level consistent with that at some reasonable depth into the subgrade (possibly 18 inches or more) right after the specimen has been conditioned by a minimum of 1000 load cycles at the same stress level. It is believed that the inverse of the resilient modulus may also be used for the subgrade with adequate accuracy. The plastic deformations will accrue from the ALPHA(3) and GNU(3) input.

The creep compliance values are input as a one-dimensional array to VESYS IIM as "LAYER1", "LAYER2" or "LAYER3" (depending on actual material layer counting from the top) and must correspond to another array called "TSTATIC", which gives the appropriate values of time in seconds. The "LAYER" values and "TSTATIC" values (same for all "LAYERS") represent a set of points from that layer's creep compliance curve. These points are used to define layer stiffness.

Asphaltic Concrete Surface Material for Layer 1

Various master creep compliance curves at a reference temperature of 70°F for asphaltic concrete have been accumulated from the

literature. Other data appeared at different temperatures with appropriate time-temperature shift factors and were converted to master curves at 70°F.

The LAYER1 values are the values of creep compliance from the master creep compliance curve for asphaltic concrete, taken to correspond to the times in the TSTATIC array. Figure 25 shows a plot of the various curves considered and the average curve derived. Table 7 provides the individual values used for the six curves used to calculate the LAYER1 average curve. These data were primarily derived from three sources as identified in Table 7.

Asphaltic Concrete Base Material

Using a three-layer system, it is likely that an asphalt-stabilized base would be combined with LAYER1 as there would usually be a subbase layer to be used as LAYER2, leaving LAYER3 for subgrade. Fig. 26 and Table 8 provide data for asphaltic-stabilized base materials.

Clay Subgrade for LAYER3

Data from three curves available in the literature appear in Table 9. It is believed that the stress levels for all of these curves would far exceed that experienced in the subgrade at some depth that would represent "the zone of mean effect", which is meant to define a depth at which the effects of loading on the subgrade are at an average. Stress and strains in the subgrade decrease with depth, becoming small numbers after say 8 to 12 feet. In one case studied, involving a thick pavement section, average effects occurred around 18 inches. With a thinner section, the depth should be greater.

Dynamic Creep Compliance Tests

The use of a static creep compliance test as the sole means of defining layer stiffness appears questionable as it is an established fact that the elastic moduli obtained from static single-load tests vary greatly from those obtained from repetitive loading dynamic tests after many cycles of loading. Both resilient and viscous responses generally decrease with repetitive loading until some number of cycles at which they become stabilized, the resilient responses after relatively few cycles (usually 200 to 1000) and the viscous effects after many cycles (possibly more than 1,000,000). While several parameters affect the test results for a single material (lateral and vertical stress levels, stress history, duration and frequency of loading, etc.), the most dominant cause for these differences may be stress history or conditioning.

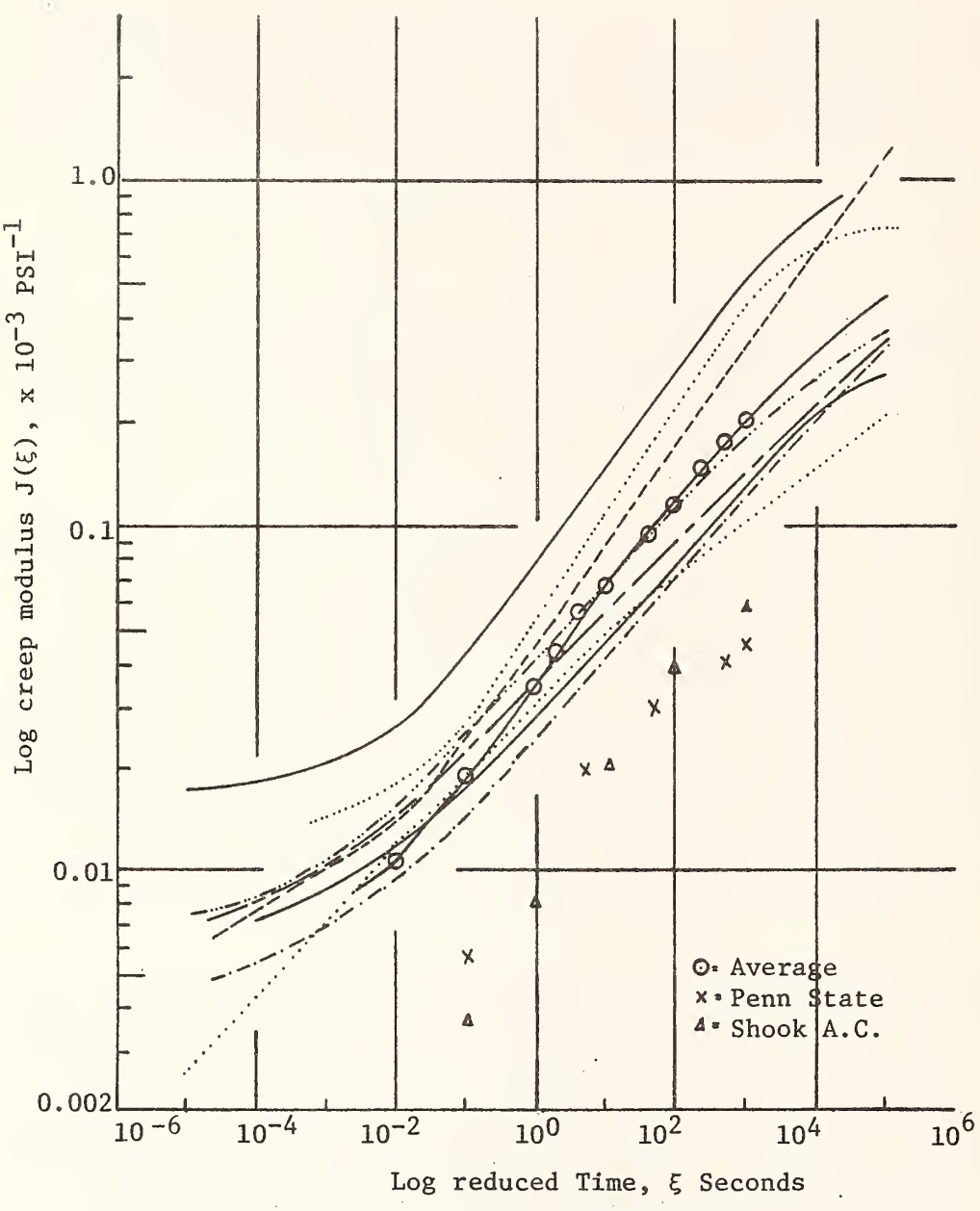

Figure 25. Master curves for unconfined compressive creep at 70°F, initial samples.

Table 7. Data for master creep compliance curves (LAYER1), asphaltic concrete, 70°F

Load Duration (Sec)	Utah Test Section			
	SECTION 12	SECTION 10	SECTION 2	SECTION 4
.01	1.5×10^{-5}	1.2×10^{-5}	1.4×10^{-5}	2.6×10^{-5}
.1	2.2×10^{-5}	1.9×10^{-5}	2.3×10^{-5}	4.2×10^{-5}
1	3.5×10^{-5}	3.0×10^{-5}	4.5×10^{-5}	8.0×10^{-5}
3	4.5×10^{-5}	3.5×10^{-5}	6.6×10^{-5}	9.5×10^{-5}
5	5×10^{-5}	4.2×10^{-5}	7.9×10^{-5}	1.3×10^{-4}
10	5.5×10^{-5}	4.9×10^{-5}	9.0×10^{-5}	1.6×10^{-4}
50	8×10^{-5}	6.5×10^{-5}	1.4×10^{-4}	2.1×10^{-4}
100	9×10^{-5}	7.0×10^{-5}	1.7×10^{-5}	2.8×10^{-4}
250	1.15×10^{-4}	7.5×10^{-5}	2.0×10^{-4}	3.2×10^{-4}
500	1.25×10^{-4}	9.1×10^{-5}	2.8×10^{-4}	4.0×10^{-4}
750	1.3×10^{-4}	9.9×10^{-5}	3.0×10^{-4}	4.8×10^{-4}
1000	1.4×10^{-4}	1.1×10^{-4}	3.3×10^{-4}	5.1×10^{-4}
% A.C.	4.9%	6.0%	5.3%	6.4%
Visc.	Low	Low	High	High

Table 7. Data for master creep-compliance curve (LAYER1), asphaltic concrete, 70 F (con't.)

SEC	PENN STATE	LOW SHOOK AC.	LAYER I AVERAGE
.01	1.5×10^{-6}	1.7×10^{-6}	1.17×10^{-5}
.10	5.5×10^{-6}	3.5×10^{-6}	1.92×10^{-5}
1	1.44×10^{-5}	8.2×10^{-6}	3.54×10^{-5}
3	1.7×10^{-5}	1.3×10^{-5}	4.52×10^{-5}
5	2.0×10^{-5}	1.6×10^{-5}	5.62×10^{-5}
10	2.35×10^{-5}	2.1×10^{-5}	6.64×10^{-5}
50	3.0×10^{-5}	3.3×10^{-5}	9.30×10^{-5}
100	3.02×10^{-5}	3.9×10^{-5}	1.13×10^{-4}
250	3.7×10^{-5}	4.5×10^{-5}	1.32×10^{-4}
500	4.0×10^{-5}	5.0×10^{-5}	1.64×10^{-4}
750	4.3×10^{-5}	5.4×10^{-5}	1.84×10^{-4}
1000	4.40×10^{-5}	5.8×10^{-5}	1.99×10^{-4}
10,000	4.60×10^{-5}		

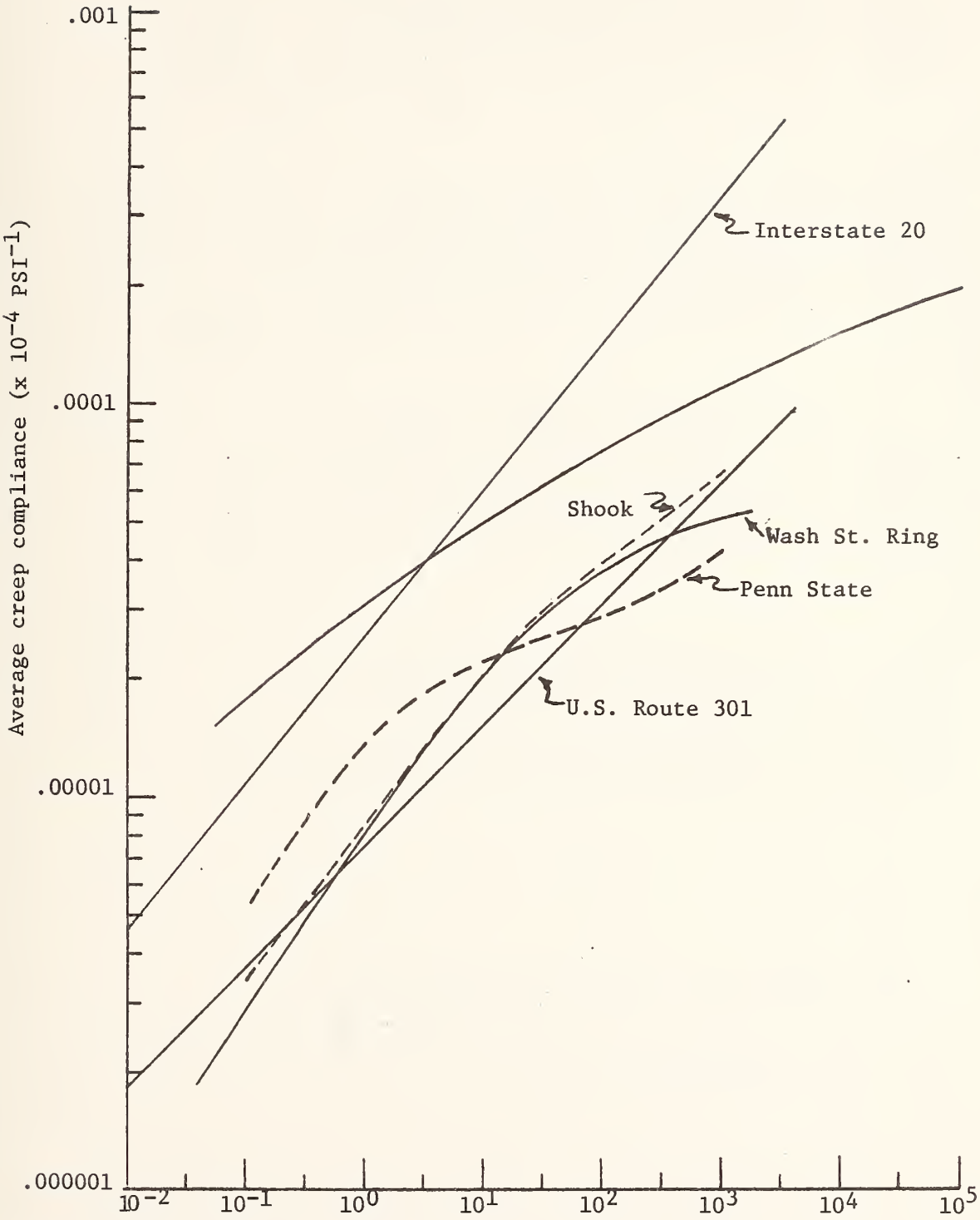

Figure 26. Creep compliance master curve plots for various A.C. mixtures

Table 8. Values for Master creep compliance curves material, asphaltic concrete base material

SEC.	WASHINGTON ST. AVERAGE	IH 20 AVERAGE	U.S. ROUTE 301 AVERAGE	LAYER 1 AVERAGE
.01	2.8×10^{-6}	1.1×10^{-5}	3.8×10^{-6}	5.87×10^{-6}
.10	5.1×10^{-6}	1.7×10^{-5}	5.4×10^{-6}	9.17×10^{-6}
1	8.5×10^{-6}	2.6×10^{-5}	7.8×10^{-6}	1.41×10^{-5}
3	1.12×10^{-5}	3.4×10^{-5}	9.6×10^{-6}	1.83×10^{-5}
5	1.34×10^{-5}	4.05×10^{-5}	1.1×10^{-5}	2.16×10^{-5}
10	2.0×10^{-5}	6.3×10^{-5}	1.6×10^{-5}	3.3×10^{-5}
50	2.85×10^{-5}	9.7×10^{-5}	2.3×10^{-5}	4.95×10^{-5}
100	3.8×10^{-5}	1.5×10^{-4}	3.3×10^{-5}	7.37×10^{-5}
250	4.2×10^{-5}	1.85×10^{-4}	3.9×10^{-5}	8.87×10^{-5}
500	4.6×10^{-5}	2.3×10^{-4}	4.7×10^{-5}	1.08×10^{-4}
750	5.0×10^{-5}	2.9×10^{-4}	5.6×10^{-5}	1.32×10^{-4}
1000	5.2×10^{-5}	3.6×10^{-4}	6.8×10^{-5}	1.6×10^{-4}
10000				

Table 9. Creep Compliances Curves for Clay Subgrades

SEC.	SHOOK 1	SHOOK 2	HIGH PENN	LAYER 3 AVERAGE
.01	2.83×10^{-4}	3.0×10^{-4}	2.2×10^{-4}	2.68×10^{-4}
.1	2.95×10^{-4}	3.19×10^{-4}	2.8×10^{-4}	2.98×10^{-4}
1	3.17×10^{-4}	3.3×10^{-4}	4.2×10^{-4}	3.56×10^{-4}
3	3.2×10^{-4}	3.37×10^{-4}	4.5×10^{-4}	3.69×10^{-4}
5	3.25×10^{-4}	3.4×10^{-4}	4.8×10^{-4}	3.82×10^{-4}
10	3.35×10^{-4}	3.45×10^{-4}	5.0×10^{-4}	3.93×10^{-4}
50	3.42×10^{-4}	3.52×10^{-4}	5.6×10^{-4}	4.18×10^{-4}
100	3.5×10^{-4}	3.63×10^{-4}	5.9×10^{-4}	4.34×10^{-4}
250	3.55×10^{-4}	3.69×10^{-4}	6.2×10^{-4}	4.48×10^{-4}
500	3.6×10^{-4}	3.75×10^{-4}	6.4×10^{-4}	4.58×10^{-4}
750	3.65×10^{-4}	3.78×10^{-4}	6.6×10^{-4}	4.68×10^{-4}
1000	3.7×10^{-4}	3.81×10^{-4}	6.8×10^{-4}	4.77×10^{-4}
10000			7.0×10^{-4}	
Dry Density	104.1 pcf	98.2 pcf		
% Moisture	18.9%	23.6%		

There is little question that the structure of the material is usually "tightened up" or stiffened by repetitive loading, leading to lesser strains under a specific stress. As much the same thing is believed to happen under wheel loadings, it appears that the inverse of creep compliance D(T) (the elastic modulus used by VESYS IIM in its analysis) should approximate dynamic values of elastic modulus obtained from repetitive loading tests. In general, values of creep compliance (ε/σ) from single or several incremental creep compliance tests are considerably larger than the inverses of resilient or dynamic moduli (σ/ε) for the same materials.

It was clear that the calculated vertical and radial strains from VESYS IIM were too large, resulting in extravagant predictions of rutting and cracking. This was attributed to the high values of creep compliance obtained from static test data in the literature.

Austin Testing Engineers Inc had conducted dynamic modulus tests on six asphaltic concrete cores recently taken from AASHO Road Test pavements. The pulse used was basically a square wave with a duration of 0.1 sec. In effect this amounted to a short-term creep compliance test under dynamic conditions; i.e., after many previous cycles of loading. These tests were run at three different temperatures and at three stress levels. The output appeared in the form of the trace shown in Figure 27 for the core from section 428.

Strains were obtained at .05 sec and .1 sec as shown and D(T) calculated for each case. The value 484 is a calibration factor used to convert the measured trace height in inches to total strain in inches. The result is two points on a limited dynamic creep-compliance test.

As most all creep compliance data in the literature plots linearly over a range between around .01 and 10 secs on a plot of Log D versus Log Time, it was reasoned that a line drawn through two or three points obtained from the dynamic trace and extrapolated in each direction might provide a reasonable basis for development of a broader creep compliance relation based on dynamic test conditions. Figure 28 shows such plots at 40°, 70° and 100°F. and the resulting calculations for the time-temperature shift function BETA used in VESYS IIM.

Use of this creep compliance vector and value of BETA with otherwise average values for a section designed using the Texas Highway Department design procedures produced the most reasonable responses seen up to that time from a VESYS IIM solution.

This approach was discussed in detail with Mr. Kenis, FHWA contract manager, during his visit in February. One idea resulting was that a creep compliance test immediately following dynamic testing

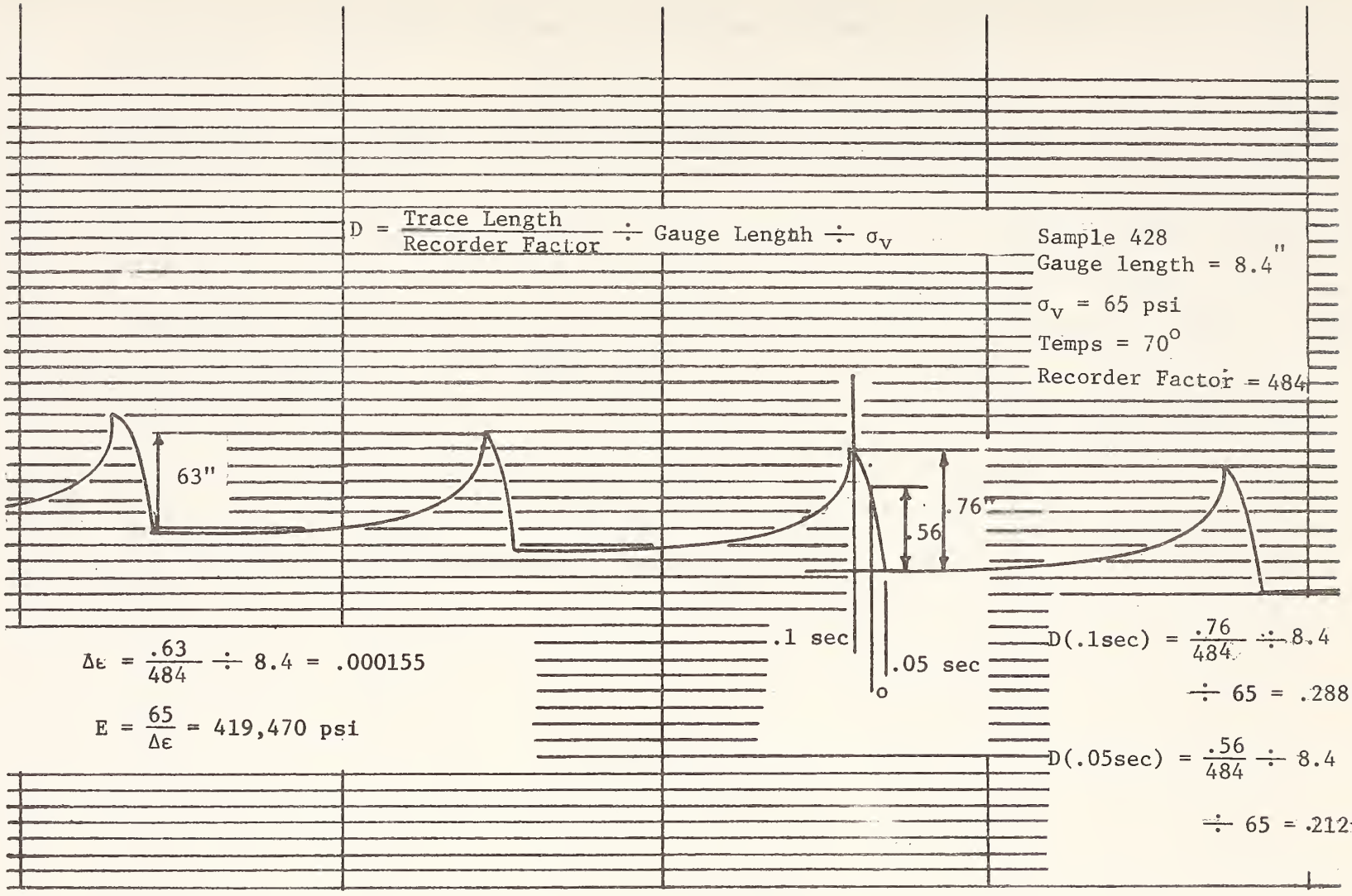
D = Trace Length / Recorder Factor ÷ Gauge Length ÷ σv
Sample 428
Gauge length = 8.4"
σv = 65 psi
Temps = 70°
Recorder Factor = 484
63"
.76"
.56
.1 sec
.05 sec
Δε = .63/484 ÷ 8.4 = .000155
E = 65/Δε = 419,470 psi
D(.1sec) = .76/484 ÷ 8.4
÷ 65 = .288 x 10-5
D(.05sec) = .56/484 ÷ 8.4
÷ 65 = .212 x 10-5

Figure 27. Dynamic trace and calculations.

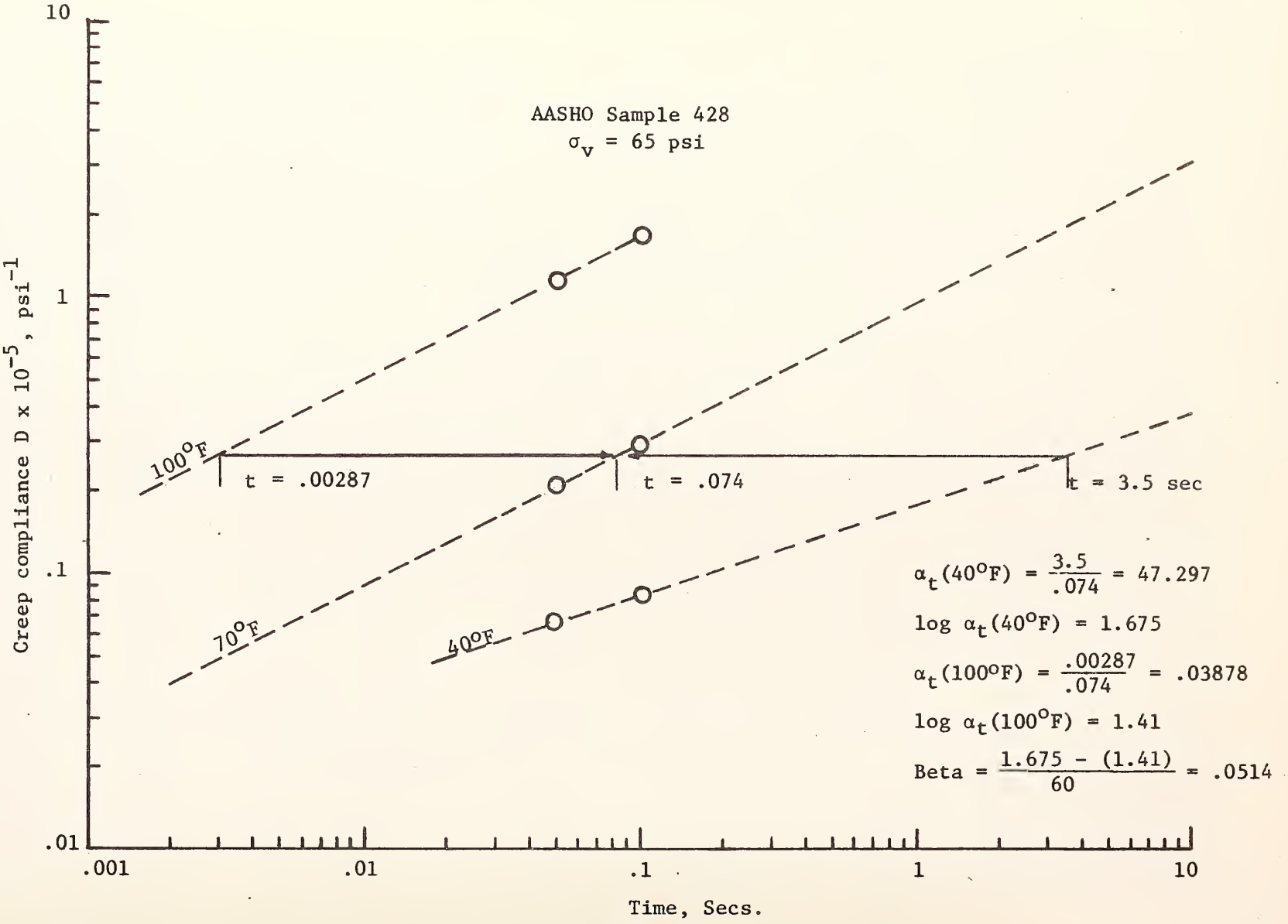

Figure 28. Dynamic creep compliance curves based on times .05 and .10 secs during a single cycle of .10 sec.

should be sufficiently conditioned to provide creep compliance values more representative of field conditions than the usual static test. It was decided that the four tests on AASHO cores to be conducted for this project to produce values for ALPHA(1) and GNU(1) would include a creep compliance test before and one after the dynamic test. It was understood that this would not be conclusive, however, as these samples have a stress history of years of wheel loads and the difference should be much less than for the usual new samples compacted in the laboratory.

Figure 29 provides the 70°F creep compliance curves developed as previously discussed for six test sections from the AASHO Road Test pavements. Note that curvature has been arbitrarily added as dashed portions of lines to better follow trends observed in the literature. The curve with "X's" for points represents the mean curve. Table 10 provides values of BETA, points in the creep-compliance-time vectors for input to VESYS IIM, including low high and mean values and their standard deviations and coefficients of variation.

It is interesting to note the variability for a situation where highway engineers did their level best to control it during construction, but the tests were generally on full-depth cores that included the original pavement and subsequent overlays. As there was variation in relative depths of the two mixes between the six cores (each from a different section), some variation must be expected. As the range in dynamic moduli is representative of the usual range in the literature at 70°F, these data should be fairly adequate for use in sensitivity analyses. The variation in this case includes "in-project" variation and some "between project" data due to the overlays using different mixes and depths.

Creep Compliance Arrays for Asphaltic Concrete

The hypothesis was drawn in the previous section that the problem was not so much the need for obtaining the data during a dynamic test as that the material needs to be conditioned dynamically prior to creep compliance testing.

The most suitable way to test this hypothesis would be to prepare specimens in the laboratory (as is usually done) and run creep compliance tests before and after the dynamic testing. It is quite probable that the strains on unconditioned samples measured during the first test, which would be representative of much of the creep compliance data in the literature, would be 4 to 10 times larger than those recorded after the sample had been repetitively loaded thousands of times.

Unfortunately, the only opportunity to test this hypothesis was on cores recently taken from selected sections of the AASHO Road test

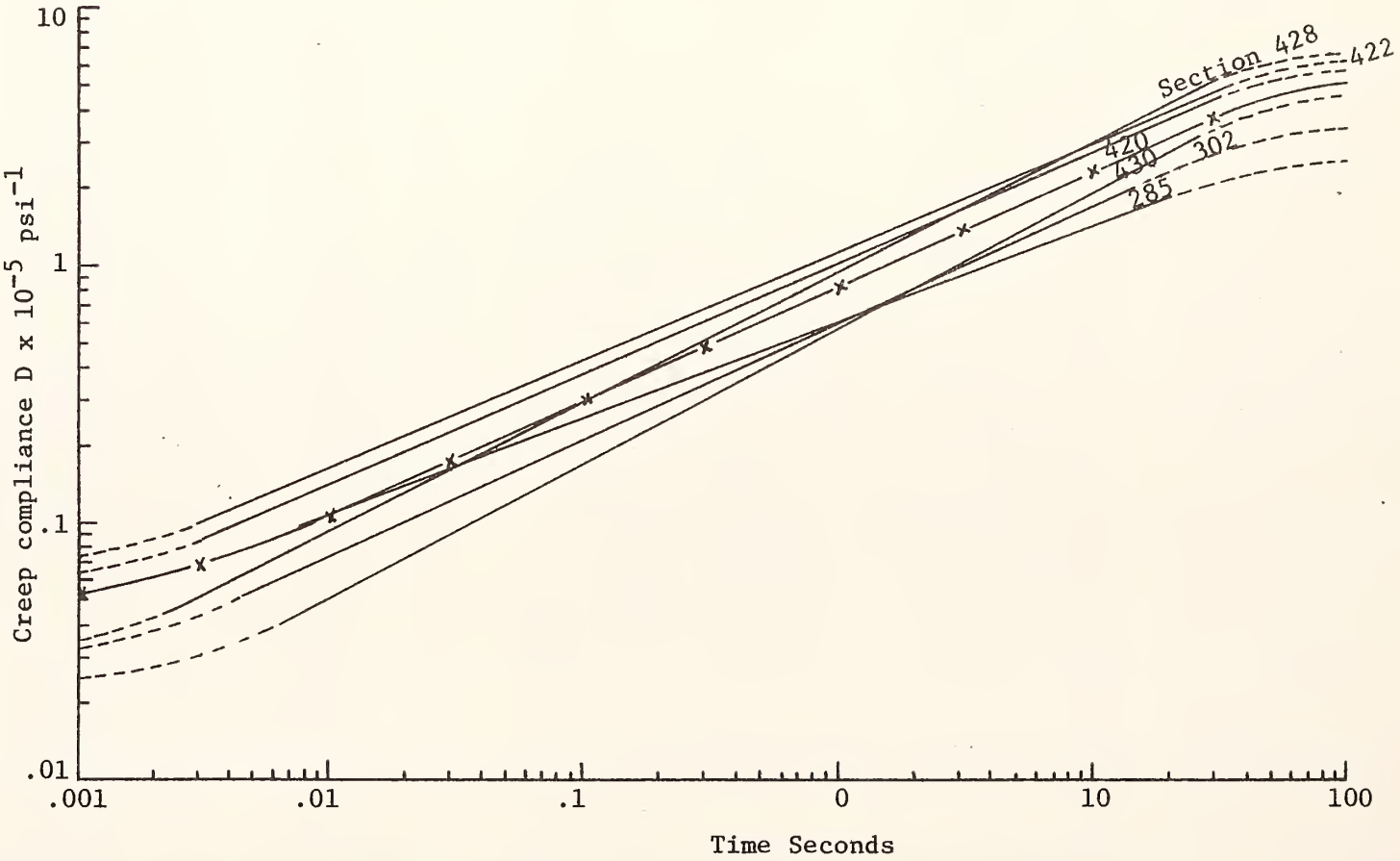

Figure 29. "Dynamic" creep compliance curves for six AASHO test section cores.

Table 10. Dynamic Creep Compliance Data from AASHO Road Test Cores

Section Number	Beta	70°F Creep Compliance X 10^{-5} psi at corresponding times in seconds											Dynamic Modulus E^* x 10^5 psi
		.001	.003	.01	.03	.1	.3	1	3	10	30	100	
286	.071	.060	.076	.112	.169	.260	.38	.60	.88	1.35	1.85	2.25	4.88
302	.047	.034	.046	.077	.124	.230	.35	.60	.96	1.65	2.55	3.60	5.22
420	.067	.077	.10	.165	.260	.430	.68	1.10	1.75	2.90	4.30	5.90	2.38
428	.051	.054	.066	.100	.160	.290	.51	.95	1.65	3.0	4.80	6.80	4.20
430	.052	.024	.031	.052	.090	.170	.30	.57	1.00	1.95	3.45	4.70	8.38
472	.036	.070	.095	.150	.240	.410	.66	1.10	1.75	3.0	4.60	6.10	3.06
Mean	.054	.053	.069	.109	.174	.30	.48	.82	1.33	2.3	3.6	4.9	4.68
Standard Deviation	.012	.019	.025	.039	.060	.094	.149	.236	.388	.682	1.09	1.57	1.92
Coeff. of Var. %	22.2	35.8	36.2	35.8	34.5	31.3	31.0	28.8	29.2	29.7	30.3	32.0	41.0
High Values	.071	.077	.100	.165	.26	.43	.68	1.10	1.75	3.0	4.8	6.8	8.36
Low Values	.036	.024	.031	.052	.09	.17	.30	.57	.88	1.35	1.85	2.25	2.38

track that have undergone wheel loading for many years prior to coring. It was expected that an important difference would be noted, but not anywhere near the scale of that probable for a newly-prepared specimen.

Four cores from sections 266, 272, 308 and 470 were tested at a temperature of 70°F and with a vertical stress of 30 psi, which was consistent with stresses expected at .55 of the layer depth predicted by elastic layer theory. The specimens were unconfined laterally and tested sequentially as follows:

1. Static creep compliance tests of 10, 100 and 1000 seconds.

2. Repetitive dynamic loading of 0.1 second on and .9 sec off with an essentially square pulse for around 15,000 cycles. Both resilient and accumulated pavement deformations were recorded.

3. Static creep compliance tests of 10, 100 and 1000 seconds.

The creep compliance results from these tests are shown in Figures 30 through 34. All plots are on log-log scales except the semi-log plot of Figure 30, which was used for clarity in displaying typical differences in results for creep compliance tests before and after dynamic loading. As expected, the "before curves" gave higher strains than the "after" tests, except in one unexplained case.

It can be seen that the "before" tests showed the effects of conditioning from each test by reduced strain with time for subsequent tests. As expected, the conditioned "after" samples tended to follow the same path. Measurement error for such small strains could account for much if not all of the minor variation in path.

It can also be seen that the variation between before and after values was much less early in the test (say 1 second) than after 10 or more seconds. This implies that the resilient response was less affected by the conditioning than the viscous response.

Figure 35 is a plot of the creep compliance curves developed for use in solutions for comparisons of calculated to measured responses for the AASHO Test Roads developed from the previous figures. Figure 36 is the same as Figure 35 except that the curves developed from the dynamic traces for other AASHO cores have been superimposed in solid lines.

Comparisons of the creep compliance curves from the dynamic traces and those from standard tests after dynamic conditioning should be made with the following differences in mind:

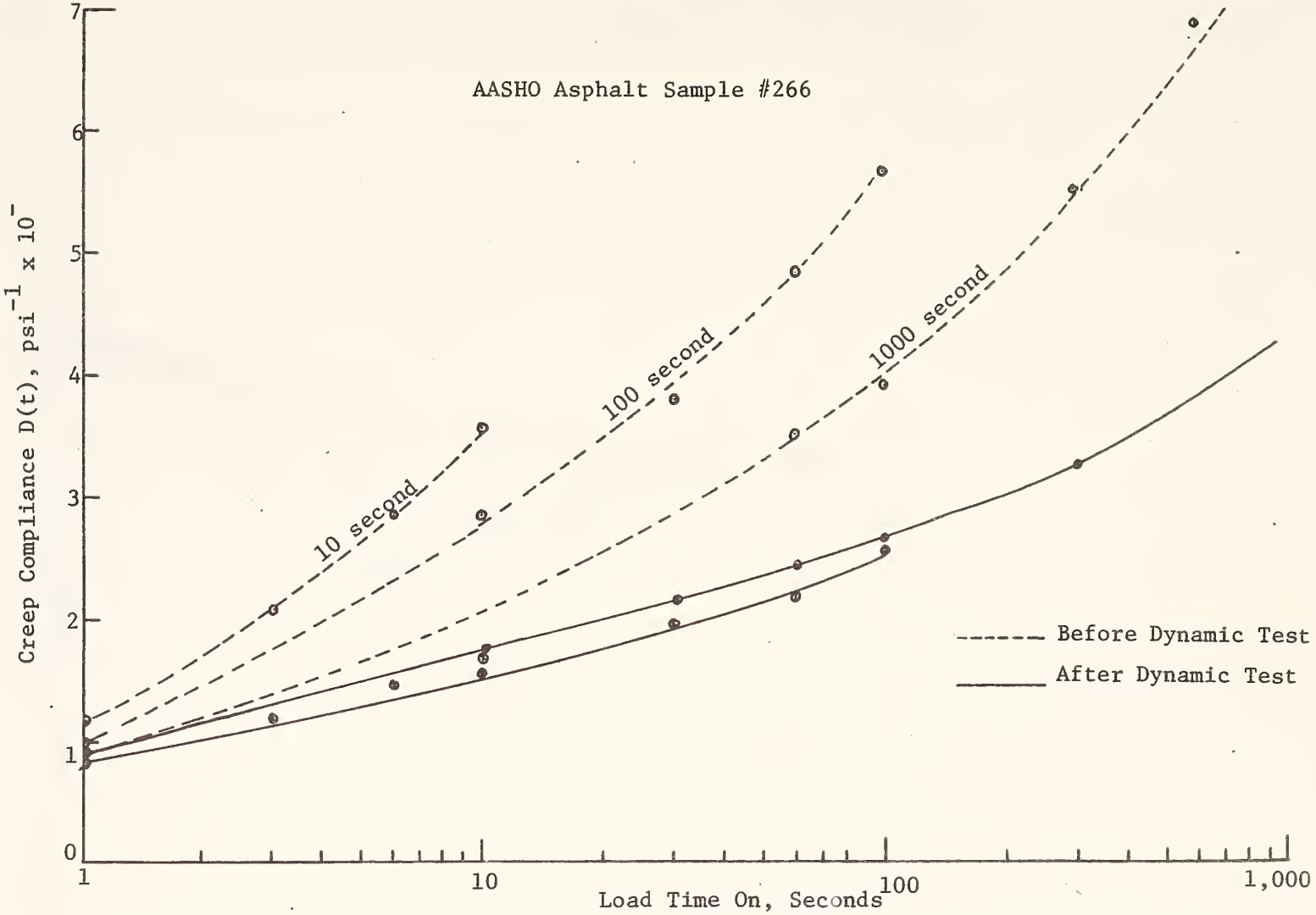

Figure 30. Creep compliance for AASHO Core 266 before and after repetitive load testing.

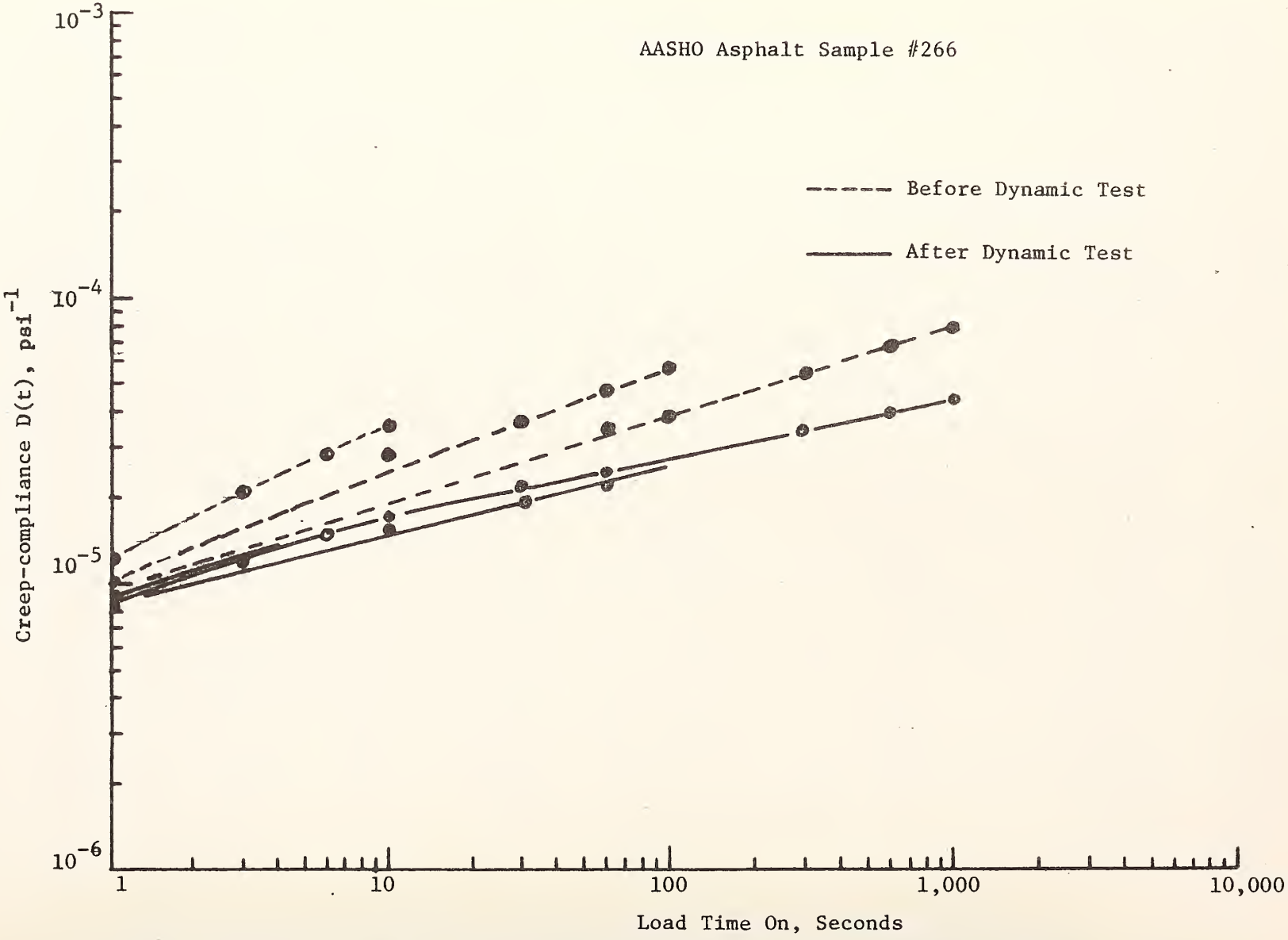

Figure 31. Creep compliance for AASHO core 266, before and after repetitive load testing.

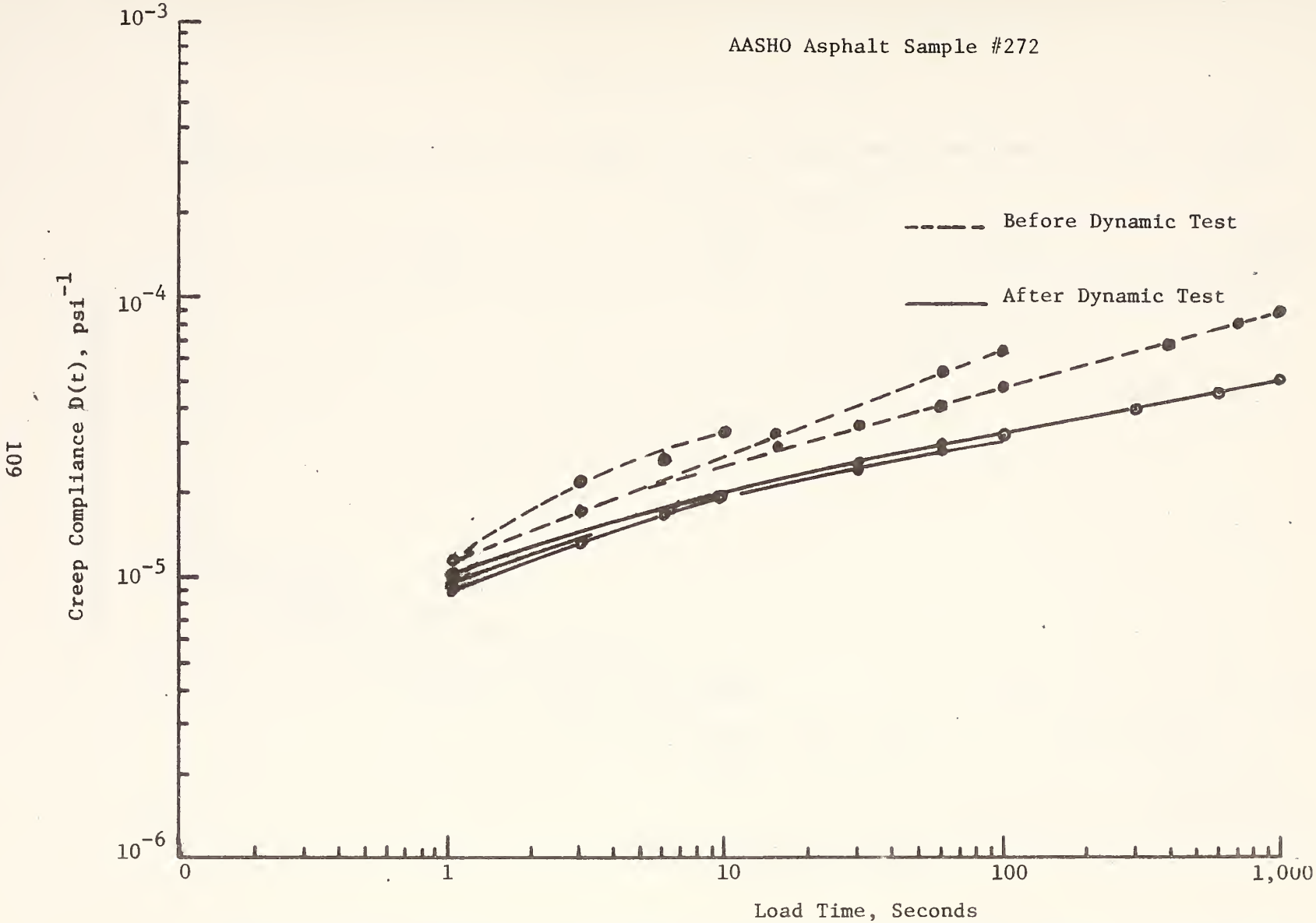

Figure 32. Creep compliance for AASHO core 272, before and after repetitive load testing.

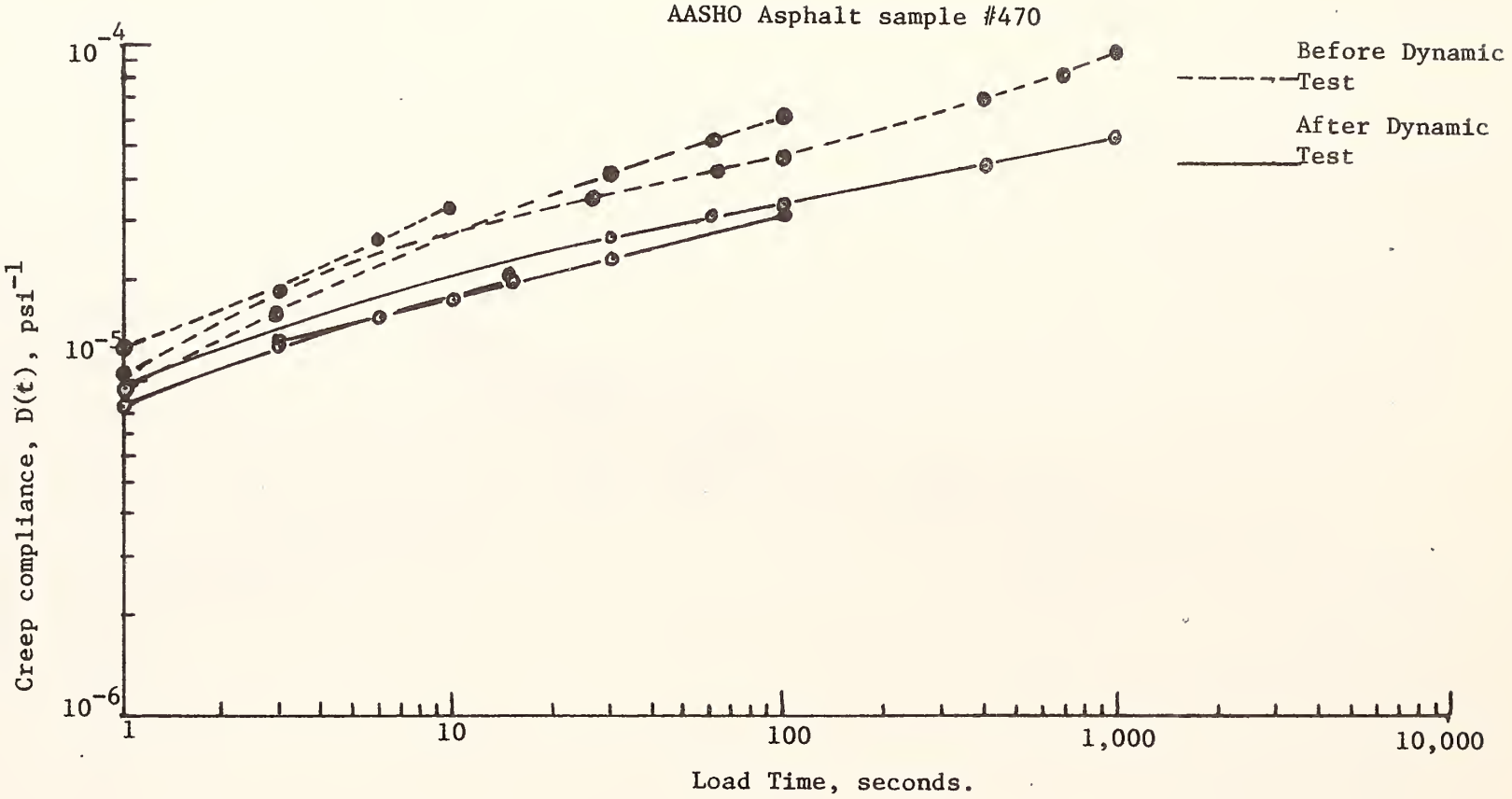

Figure 33. Creep compliance for AASHO core 470, before and after repetitive load testing.

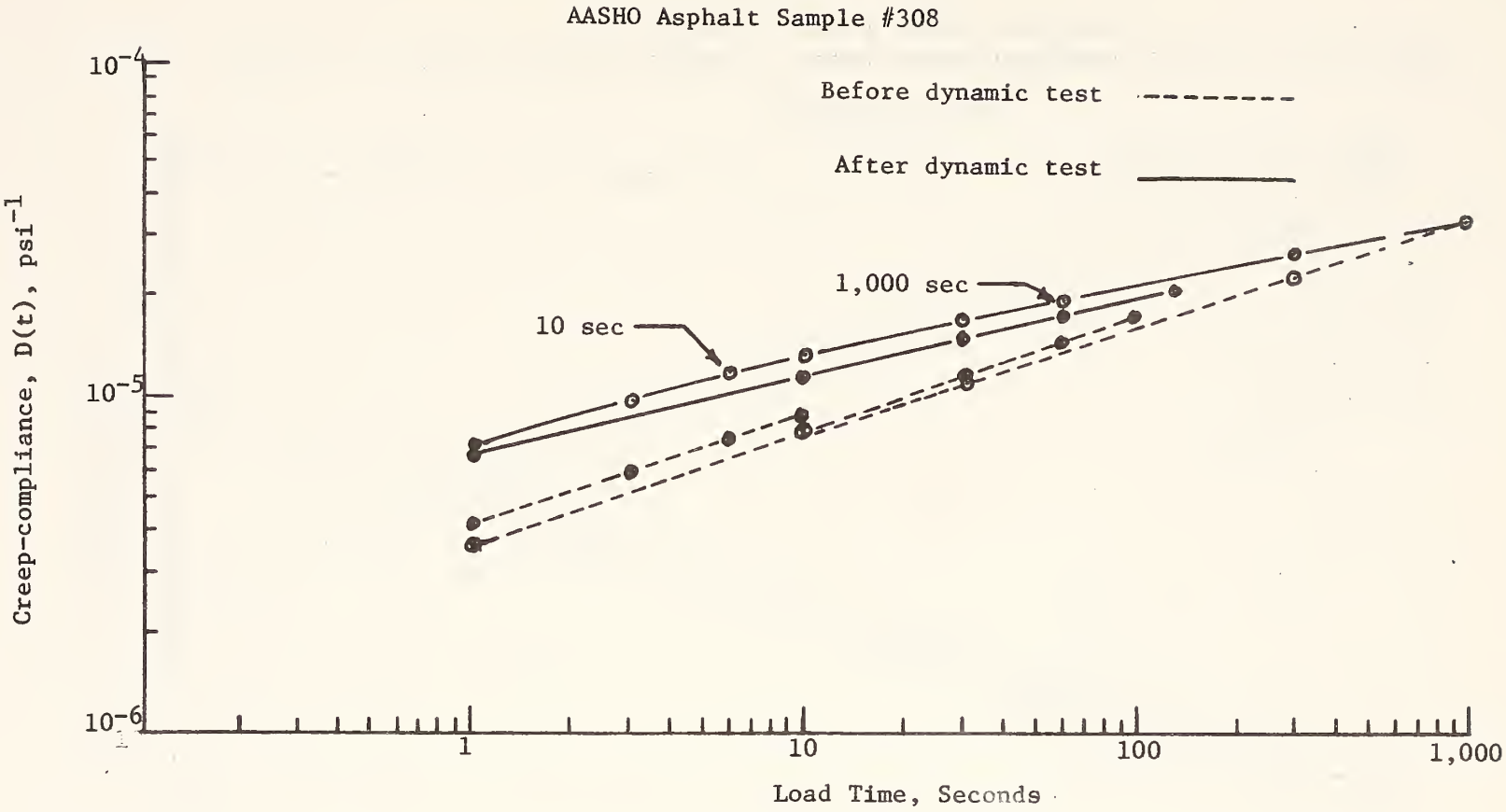

Figure 34. Creep compliance for AASHO core 308, before and after repetitive load testing.

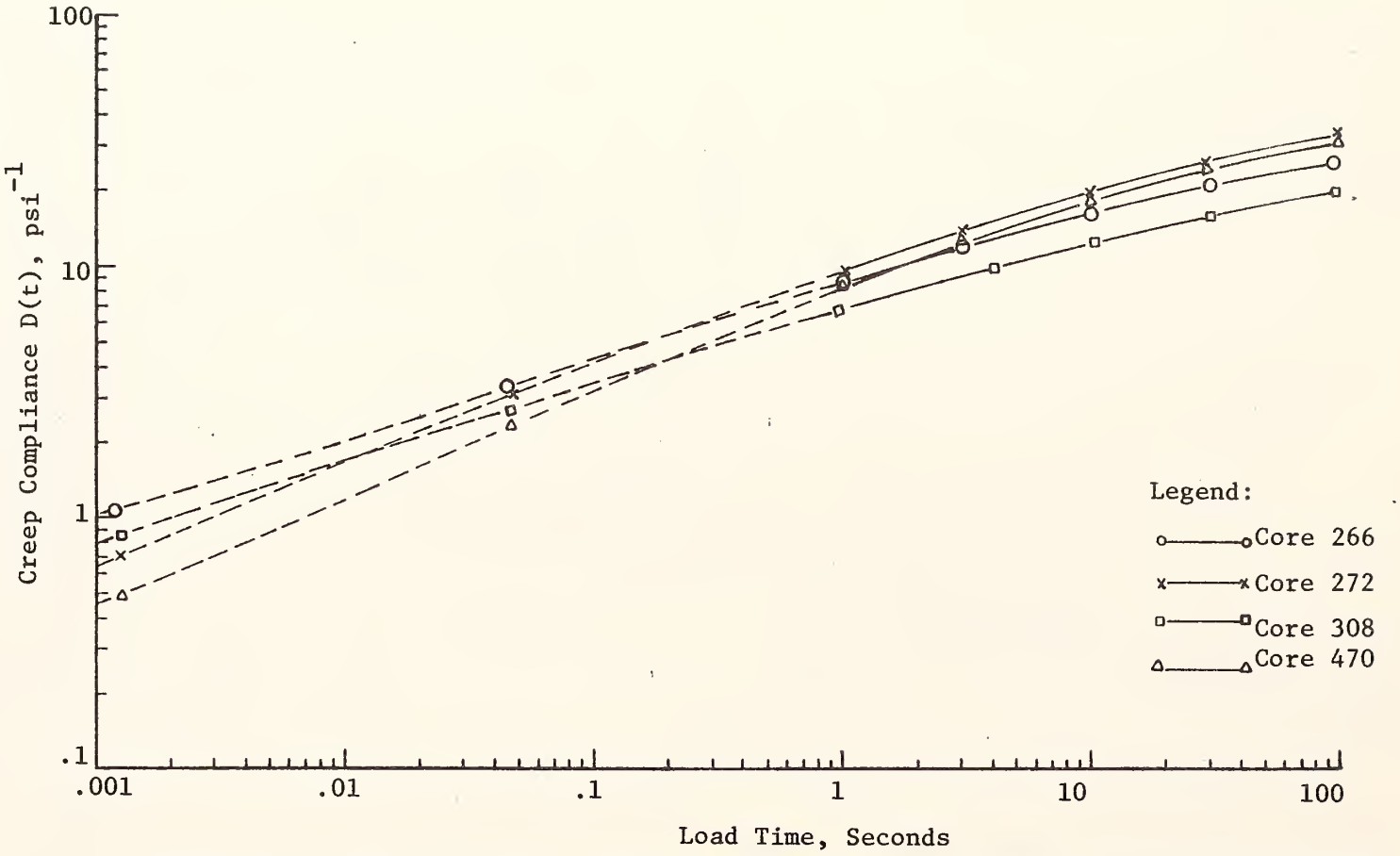

Figure 35. Creep compliance curves for AASHO cores developed from tests made after 15,000 or more dynamic load cycles.

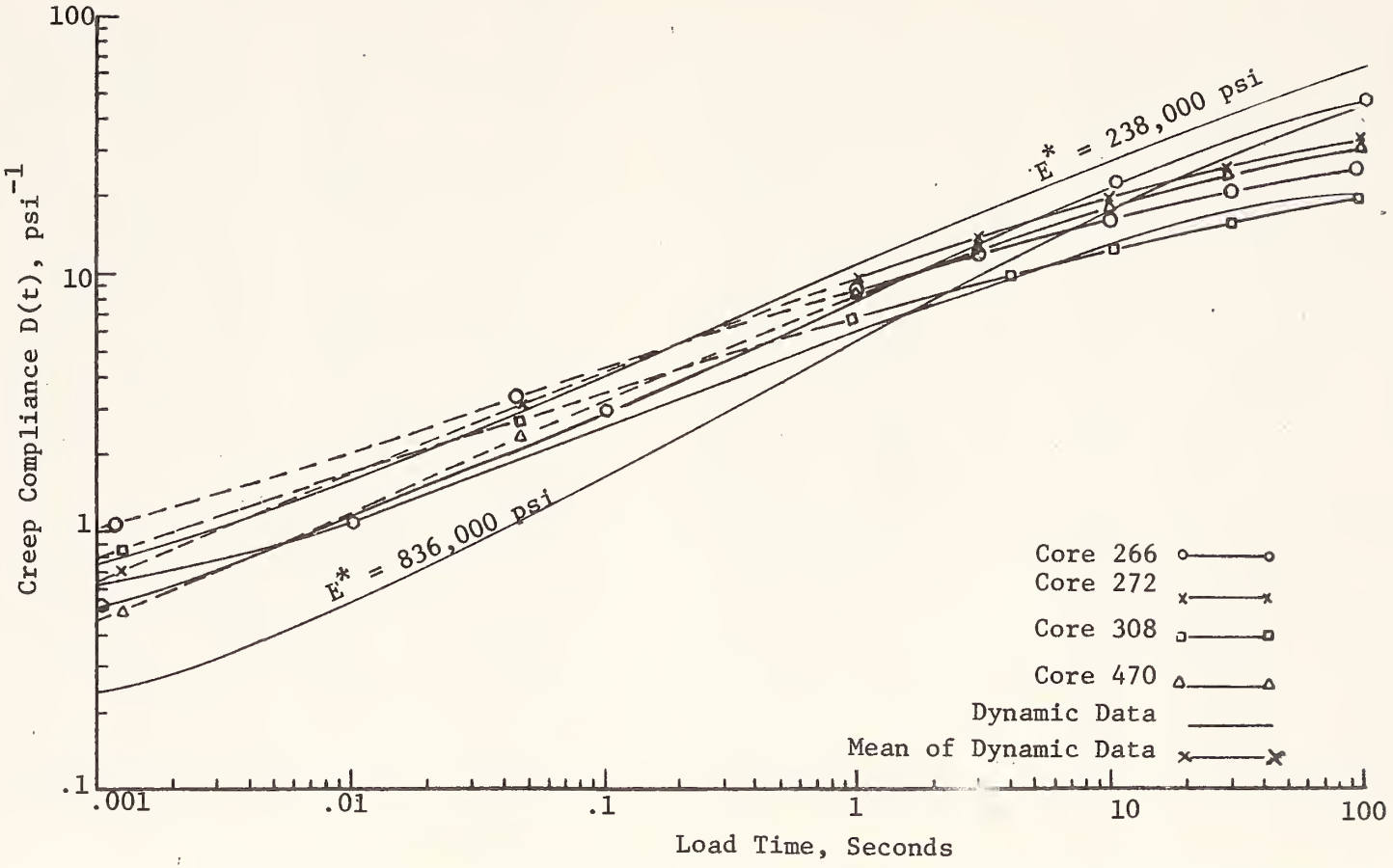

Figure 36. Creep compliance data from dynamic traces and from creep compliance tests after 15,000 or more dynamic loads.

1. The curves from dynamic traces were taken at 65 psi of vertical pressure while those from standard tests after dynamic conditioning were at 30 psi. The difference that might be expected would be a stiffer response from conditioning at a higher stress; however, this is apparently not the case. The reason may be that conditioning for the dynamic modulus tests was only until the resilient strains stabilized at around 200 cycles for 65 psi. (The test regime was 1000 cycles conditioning at 30 psi, 200 more cycles at 30 psi and measure strains, 200 cycles at 65 psi and measure strains and 200 cycles at 100 psi and measure strains). As can be seen, 30 psi data does exist and further comparisons could be made if considered worthwhile.

2. The variations in dynamic modulus were only from 283,000 psi to 357,000 psi for the 4 cores tested for creep compliance, while the variation was from 238,000 psi to 836,000 psi for the dynamic modulus tests. Note on Figure 31 that these curves are identified and represent the extremes also for the creep compliance curves at times under 1 second. It is also interesting to note that the particular core with the high dynamic modulus and low creep compliance under 1 second also displayed relatively high viscous flow as time increased.

The most important conclusions to be drawn are:

1. Both the creep compliance test after dynamic conditioning or extraction from dynamic test traces will yield stiffness relatable to dynamic strains in the range .05 to .1 second of time commonly used for those tests. This means that the resultant creep compliance values are consistent with dynamic modulus test results at similar times, which implies that they are calibrated at other times to the environment of a pavement undergoing repetitive wheel loadings.

2. The primary limitation on previous static creep compliance testing has apparently been conditioning. Attempts have been made to condition specimens in various ways such as pre-loading with a static load or several static loads, but these do not really simulate the strain conditions induced by thousands of short-term wheel loads as closely as the dynamic testing regime. It would also be very difficult to decide how much static loading would best simulate the conditioning from thousands of wheel loads.

VESYS IIM utilizes the creep compliance vectors and the time-temperature shift factor BETA to arrive at suitable values of layer stiffness for use in elastic layer solutions. Using most stati data, it has overpredicted responses greatly except when the few very stiff creep compliance curves were applied (Penn State Test Track data, for instance). Table 11 has been prepared to allow comparison of the moduli of elasticity derived at 0.1 second, which may be compared with dynamic modulus values typical for 70°F.

It can readily be seen from Table 11 that the elastic moduli derived from creep compliance tests at 0.1 second duration from the Utah data are too low, most of the other static tests approach reasonable values, but are still generally low compared to dynamic moduli reported in the literature.

It can also be seen that the elastic moduli obtained from the "after dynamic tests" or from dynamic strain traces are also lower than dynamic moduli because permanent strain during one cycle is included with the resilient strain for calculation. These values, however, are much more realistic than the static test values when compared to dynamic moduli in the literature as the permanent strain during a cycle should represent only a fraction on the order of 5% to 25% of the resilient strain (dependent on stress level, temperature and other parameters).

It appears reasonable to combine the results obtained from the dynamic strain trace (if a square pulse is used) and the "after" creep compliance curve to obtain accurate data around .03 to .1 second and from 1 to 1000 seconds. This has been done in Figure 37 with the plots in Figure 30 to improve them. Figure 37 provides curves for use in comparisons of measured and calculated responses for AASHO sections.

It is necessary to develop creep compliance curves as representative as possible for use in the main sensitivity analysis. Creep compliance values appear in Table 11 for the ten AASHO cores tested dynamically. If the values of creep compliance at 0.1 second from the ten AASHO cores are considered, these new statistical data result:

$$\text{Mean } \bar{x} = 3.3 \times 10^{-6} \text{ psi}^{-1}$$

$$\text{Standard Deviation } \sigma = 0.80 \times 10^{-6} \text{ psi}^{-1}$$

$$\text{Coefficient of Variation} = 24\%$$

Table 11. Modulus of elasticity at 0.1 seconds from static and dynamic creep compliance curves at 70°F.

Static tests

Source Of Data	Creep Compliance at 0.1 Seconds	Modulus of Elasticity or Inverse of Creep Compliance
Utah Data:		
Section 2	4.2×10^{-5} psi^{-1}	23,810 psi
Section 4	2.3×10^{-5}	43,480
Section 10	1.9×10^{-5}	52,630
Section 12	2.2×10^{-5}	45,450
Penn State Test Track A. C. Base	3.5×10^{-6}	181,800
Shook - A. C. Base	3.6×10^{-6}	277,800
U. S. Route 301 Base (Average)	5.4×10^{-6}	185,200
Washington State Test Ring Base (Average)	5.1×10^{-6}	196,100
Interstate 20 Base(Average)	1.7×10^{-5}	58,820
Dynamic tests on AASHO cores		
From creep compliance tests after dynamic tests		
Core 266	4.3×10^{-6}	232,600
Core 272	4.2×10^{-6}	238,100
Core 308	3.4×10^{-6}	294,100
Core 470	3.2×10^{-6}	312,500
From strain traces during dynamic tests:		
Core 286	2.6×10^{-6}	384,600
Core 302	2.3×10^{-6}	434,800
Core 420	4.3×10^{-6}	232,600
Core 428	2.9×10^{-6}	344,800
Core 430	1.7×10^{-6}	588,200
Core 472	4.1×10^{-6}	243,900

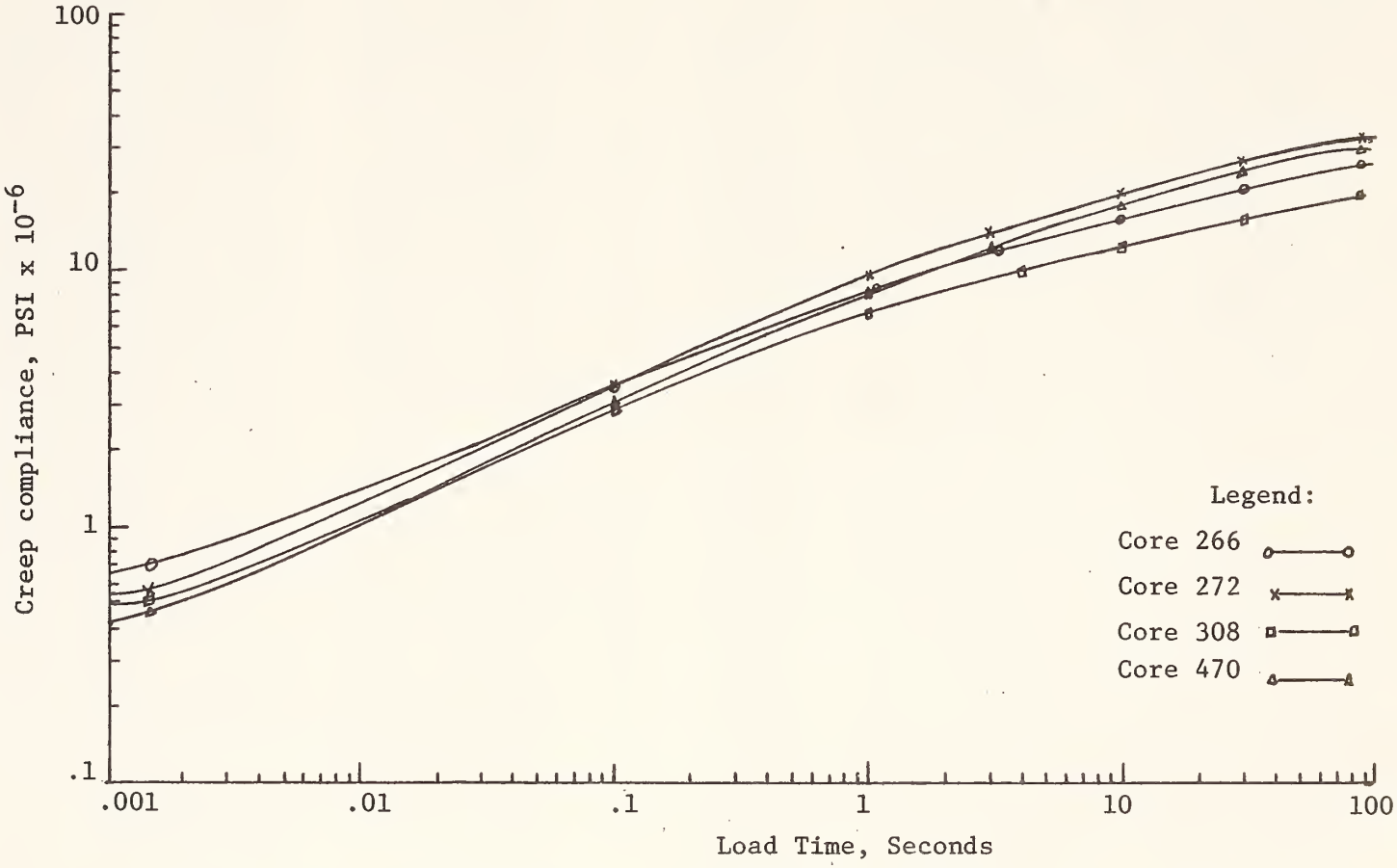

Figure 37. Creep compliance curves for AASHO cores developed from tests made after 15,000 or more dynamic load cycles.

$$\text{High value} = \bar{x} + \sigma = 4.10 \times 10^{-6} \text{ psi}^{-1}$$

$$\text{Low value} = \bar{x} - \sigma = 2.50 \times 10^{-6} \text{ psi}^{-1}$$

These values have been plotted in Figure 38 at .1 second and the remainder of the curves drawn in to approximate the shape of the curves in Figure 37. The resulting two curves designated as "High" and "Low" are those to be used in the sensitivity analysis representing a mean creep compliance curve plus and minus one standard deviation. These are believed to be quite representative of realistic conditions in a roadway having undergone some thousands of wheel loadings.

VARCOEF1 is the coefficient of variation for LAYER1. This is intended to be the coefficient of variation for a single curve (i.e., in-project variation). The mean will be taken as .25 based on study of cores from Texas highways. There are no data upon which to directly base "between-project" variation of the coefficient of variation of LAYER1, but it was assumed on the basis of judgment that the range .15 to .35 for this variable will contain 68% of the distribution to simulate one standard deviation either side of the mean.

Variations in BETA across a broad range had little effect on responses during the preliminary sensitivity analysis. Accordingly, BETA will not be varied during the main sensitivity analysis. The mean value of .054 from the six AASHO cores (see Table 10) was used.

Creep Compliance Arrays for Granular Base Materials

Dynamic creep compliance tests were run on both base and subbase materials sampled from the AASHO Road Test and compacted to densities to approximate those existing in the field. The base materials were tested at a deviator stress of 7 psi and no lateral pressure and the subbase materials at a deviator stress of 3.5 psi and a lateral pressure of 2.2 psi. These stress conditions resulted from elastic layer solutions representing approximate AASHO Road Test conditions to arrive at stress levels for testing.

Figures 39 and 40 show results of the tests on two different compacted base material samples. Tests of 10, 100, and 1000 seconds were run on each sample. It can be seen that the creep compliance values were constant and essentially the same during each test on a particular sample while the load was on. The values were fairly close between tests also. The unload curves were essentially identical for the test represented by Figure 40, indicating little permanent strain but the rate of permanent deformations was much higher for the other tests as indicated by the large increases in permanent strain between tests. Figures 41 and 42 provide similar results for tests on the subbase.

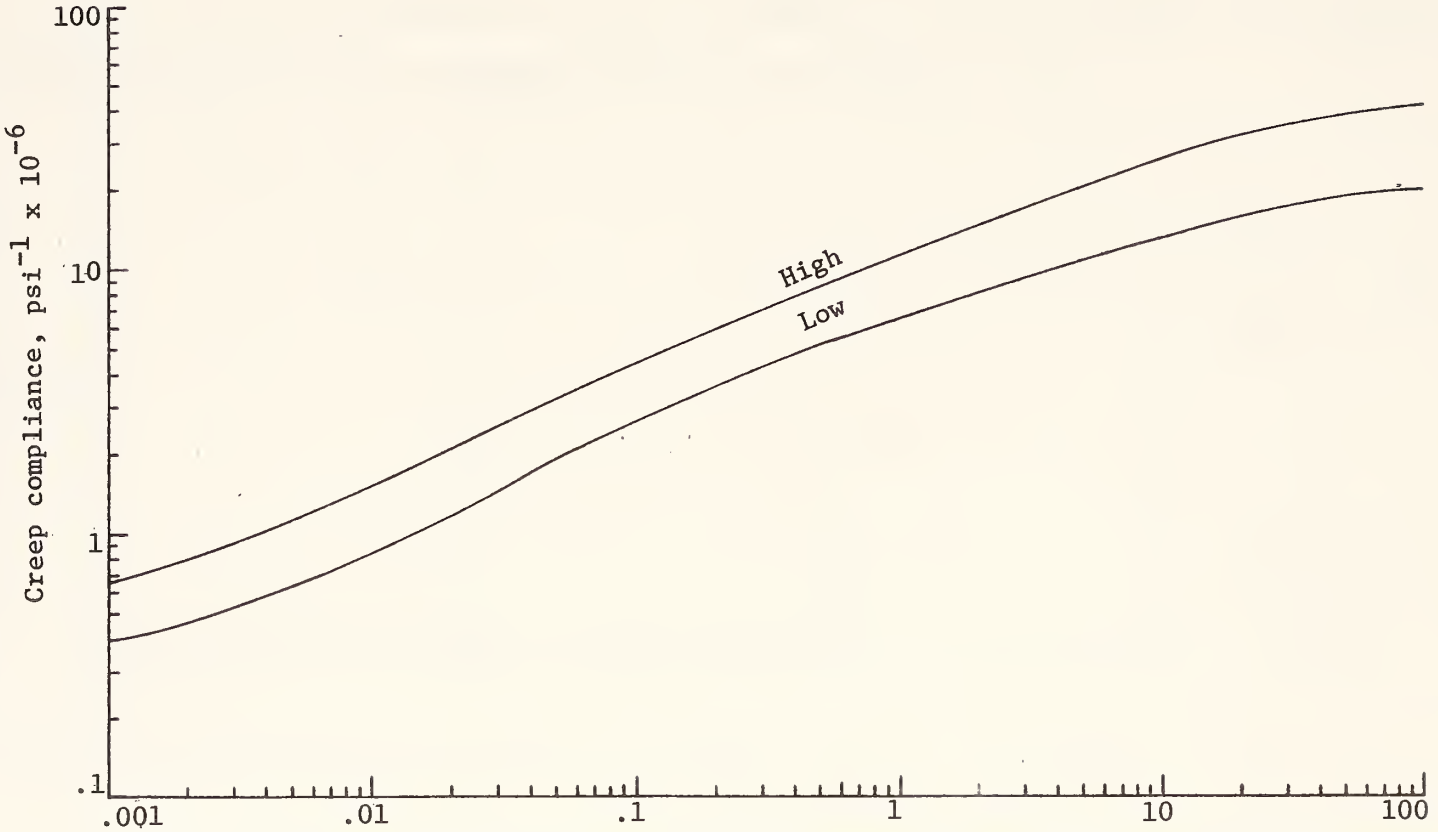

Figure 38. High and low creep compliance curves for Layer 1 for use in the sensitivity analysis (represent $\bar{x} + \sigma$ and $\bar{x} - \sigma$).

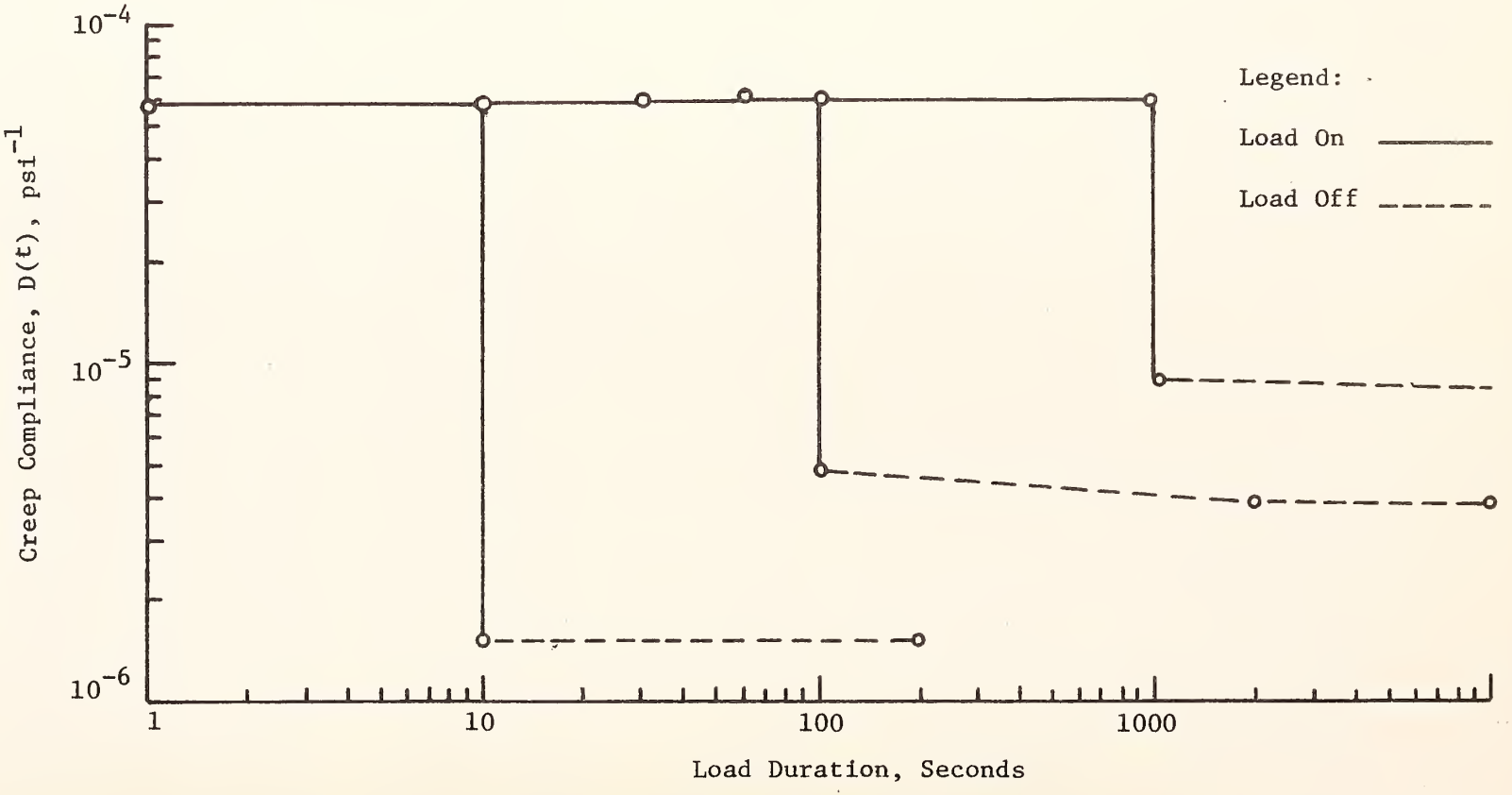

Figure 39. Creep compliance plots for three tests on sample 1, AASHO base material, $\sigma_3 = 0$, $\sigma_d = 7$ psi.

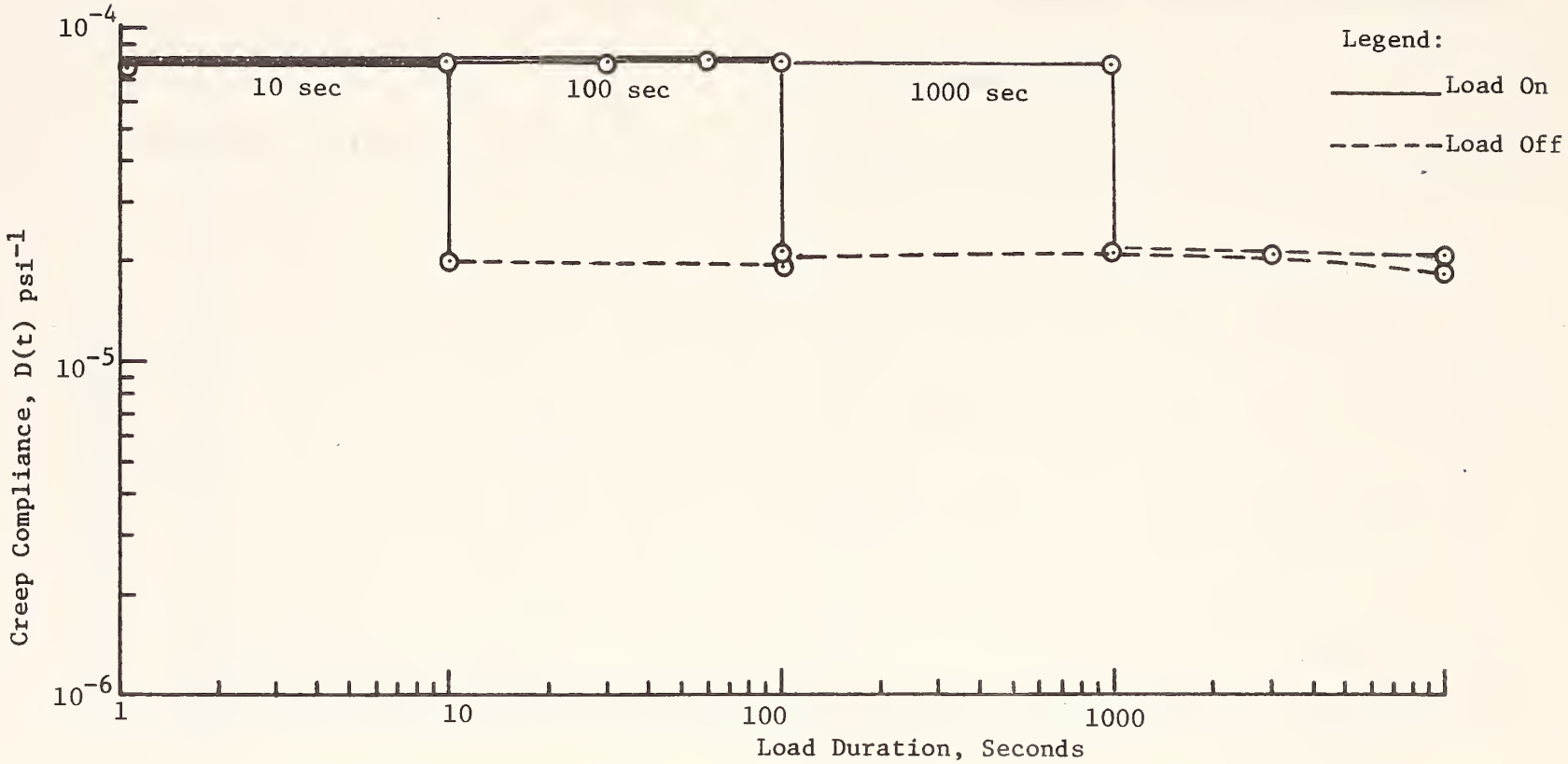

Figure 40. Creep compliance plots for three tests on Sample 3, AASHO base material, $\sigma_3 = 0$, $\sigma_d = 7$ psi.

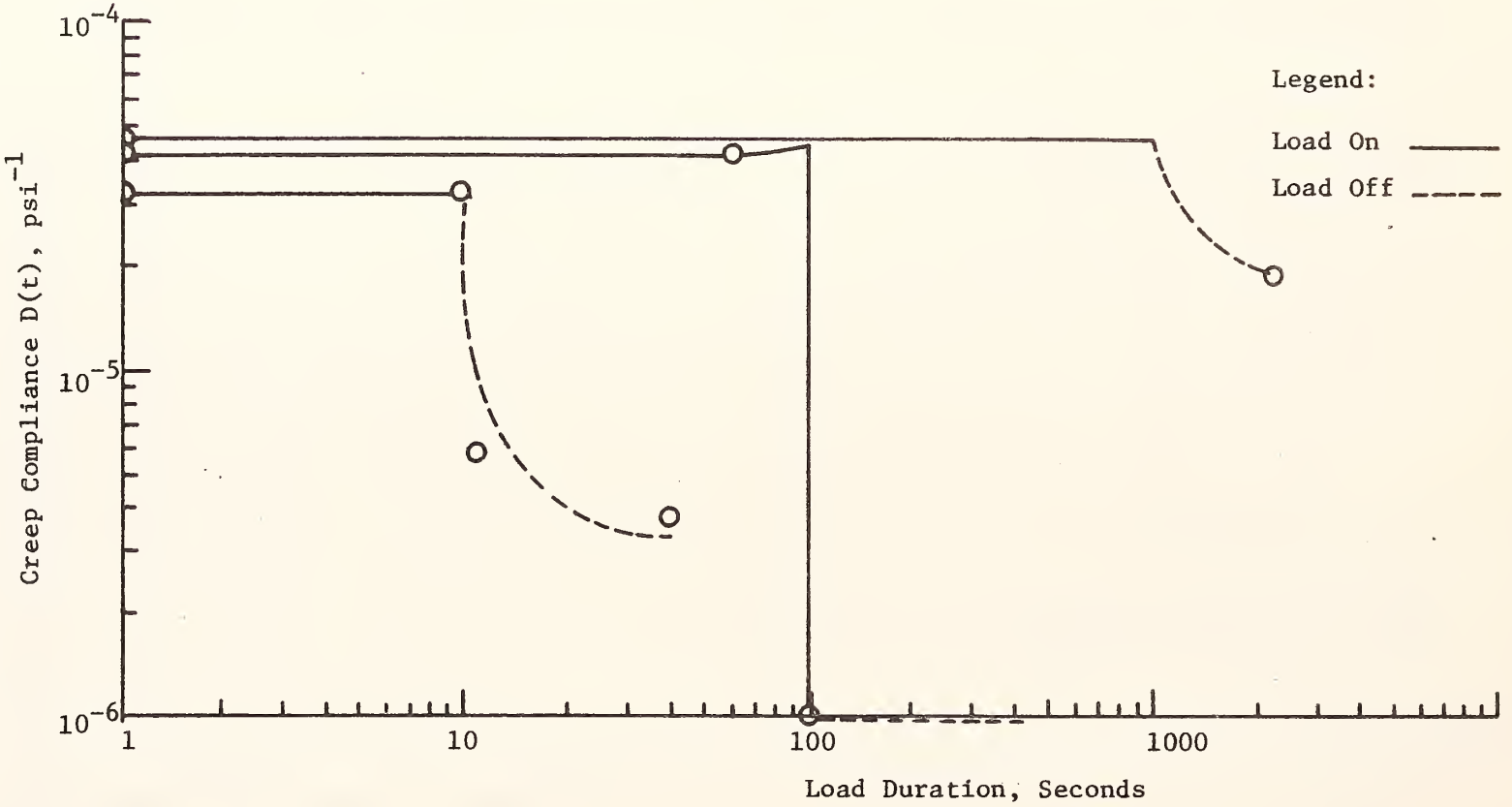

Figure 41. Creep compliance plots for three tests on Sample 1, AASHO Subbase material, σ_3 = 2.2, σ_d = 3.5 psi.

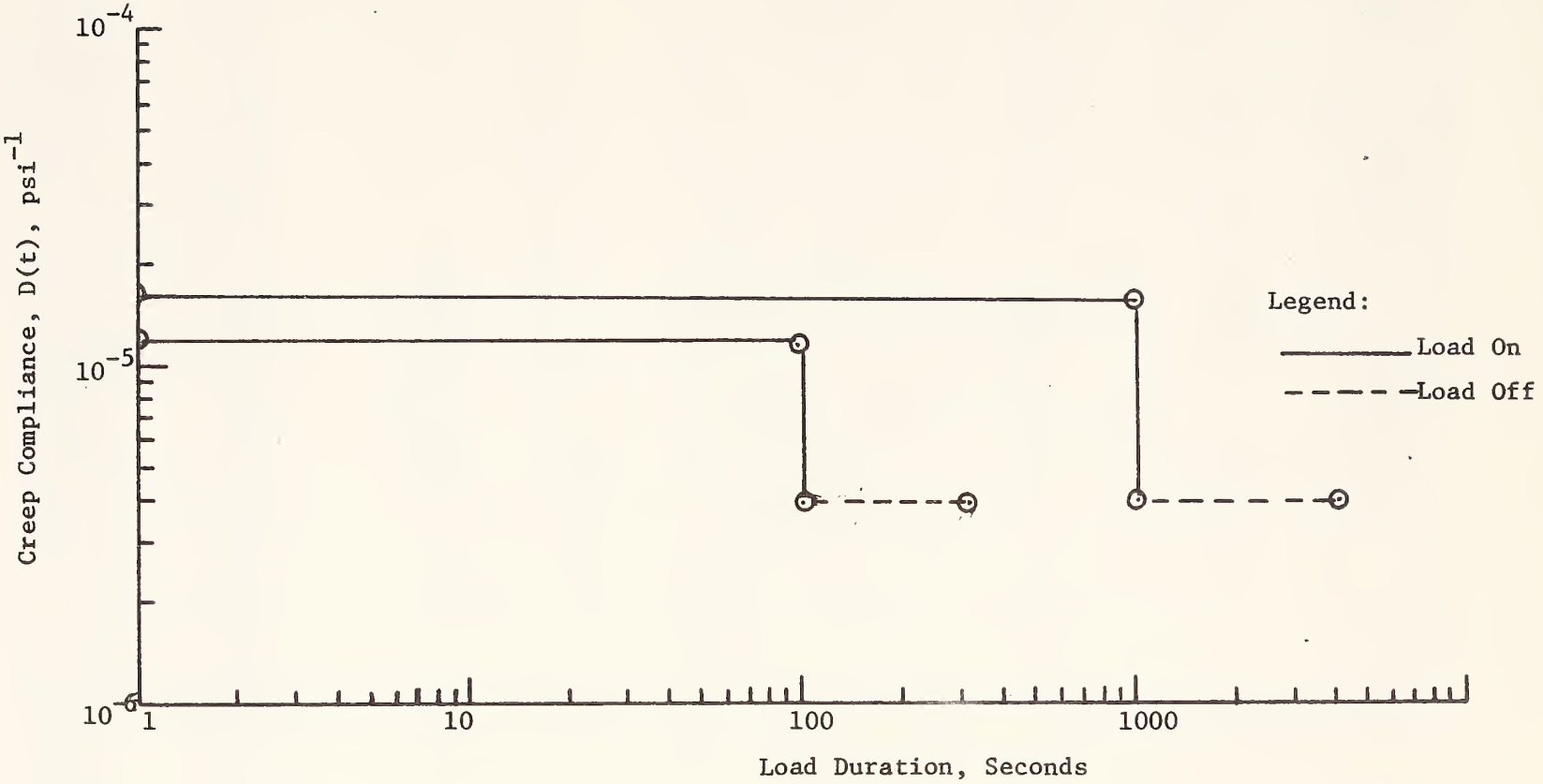

Figure 42. Creep compliance plots for three tests on Sample 3, AASHO subbase material, σ_3 = 2.2 psi, σ_d = 3.5 psi.

The important result is not the values, but the fact that the response was essentially elastic in that no discernable creep occurred during the load period. This corroborates the generally accepted hypothesis based on other testing and soil mechanics that no appreciable creep occurs in granular base or subbase materials.

As there is no creep expected with load duration, it is reasonable to use a value for creep compliance independent of time. The best characterization of a granular base material under service conditions is a single-valued creep compliance equal to the inverse of the resilient modulus obtained from dynamic testing.

The single-valued creep compliances selected for use in the main sensitivity analysis were derived from review of resilient moduli data in References 14[1] and 15[2] and other sources. These were a low of 4.55×10^{-5} and a high of 6.25×10^{-5} psi^{-1}.

The single values for creep compliance used in the prediction of performance for the AASHO Road Test sections were arrived at by averaging results of duplicate tests on the base and on the subbase materials and then combining these on a weighted basis dependent on layer thickness. The average values of resilient modulus were $\frac{1}{2}(15,750 + 19,900) = 17,800$ psi and $\frac{1}{2}(14,900 + 15,800) = 15,350$ psi for the base and subbase, respectively. The single values of creep compliance are then the inverses of the resilient moduli, 5.618×10^{-5} and 6.515×10^{-5} respectively. The equation for calculating the combined LAYER2 is then:

$$\text{LAYER2} = \frac{\text{Base Thickness (5.618)} + \text{Subbase Thickness (6.515)}}{\text{Base Thickness} + \text{Subbase Thickness}}$$

As an example, for Section 266:

$$\text{LAYER2} = 10^{-5}\left[\frac{9(5.618) + 16(6.515)}{9 + 16}\right] = 6.19 \times 10^{-5} \text{ psi}^{-1}$$

---

[1] Smith, W. S. and K. Nair, "Development of Procedures for Characterization of Untreated Granular Base Course and Asphalt-Treated Base Course Materials," Federal Highway Administration Report No. FHWA-RD-74-61, October 1973.

[2] Allen, J. J. and M. R. Thompson, "The Resilient Response of Granular Materials Subjected to Time-Dependent Lateral Stresses", a paper presented at the 53rd Annual Meeting of the Highway Research Board, January 1974.

The resulting values used were:

Section No.	Base Depth (Inches)	Subbase Depth (Inches)	LAYER2 ($PSI^{-1} \times 10^{-5}$)
266	9	16	6.19
272	9	8	6.04
308	6	12	6.22
470	6	8	6.13

Creep Compliance Arrays for Clay Subgrade

Three considerations greatly affect the approach selected for obtaining input values for creep compliance for subgrades. One is that the duration of load may be expected to be about 0.1 seconds (dependent on depth of pavement section) and is not really relatable directly to the variable DURATION, which is usually much less and is calculated to provide load duration in the surface layer. The second is that at the low stress levels experienced by the subgrade, creep occurs at such a slow rate that the change in creep compliance between say 0.08 and 0.12 seconds would not be significant. Thirdly, LAYER3 is not a very significant variable according to the preliminary sensitivity analysis. For the above reasons, it seemed more logical to use single-valued inverses of resilient moduli as for base materials to more closely simulate the stiffness used in the elastic layer solution for calculating strains and displacements. Permanent strain predictions will still accrue from the ALPHA(3), GNU(3) characterization.

The values for resilient modulus of subgrade soils were obtained through review of a variety of source material and experience in resilient modulus testing for a range of clay soils. It was decided to use resilient moduli of 5000 and 9000 psi as approximately one standard deviation above and below a mean of 7000 psi. Inversion of these values gives a low creep compliance of 11.1×10^{-5} psi^{-1} and a high of 20.0×10^{-5} psi^{-1}. These values were used in the main sensitivity analysis.

Values of LAYER3 for the predicted performance of AASHO Road Test sections were obtained from creep compliance tests after dynamic conditioning. Figure 43 shows test results for five such tests. The values of creep compliance were taken at the earliest measurable strain.

As can be seen, all of the creep compliance curves were almost flat, indicating that little creep was taking place while the load was on. This was due to relatively low deviator stresses and the fact that these subgrade soils had already been subjected to a million or more wheel loads.

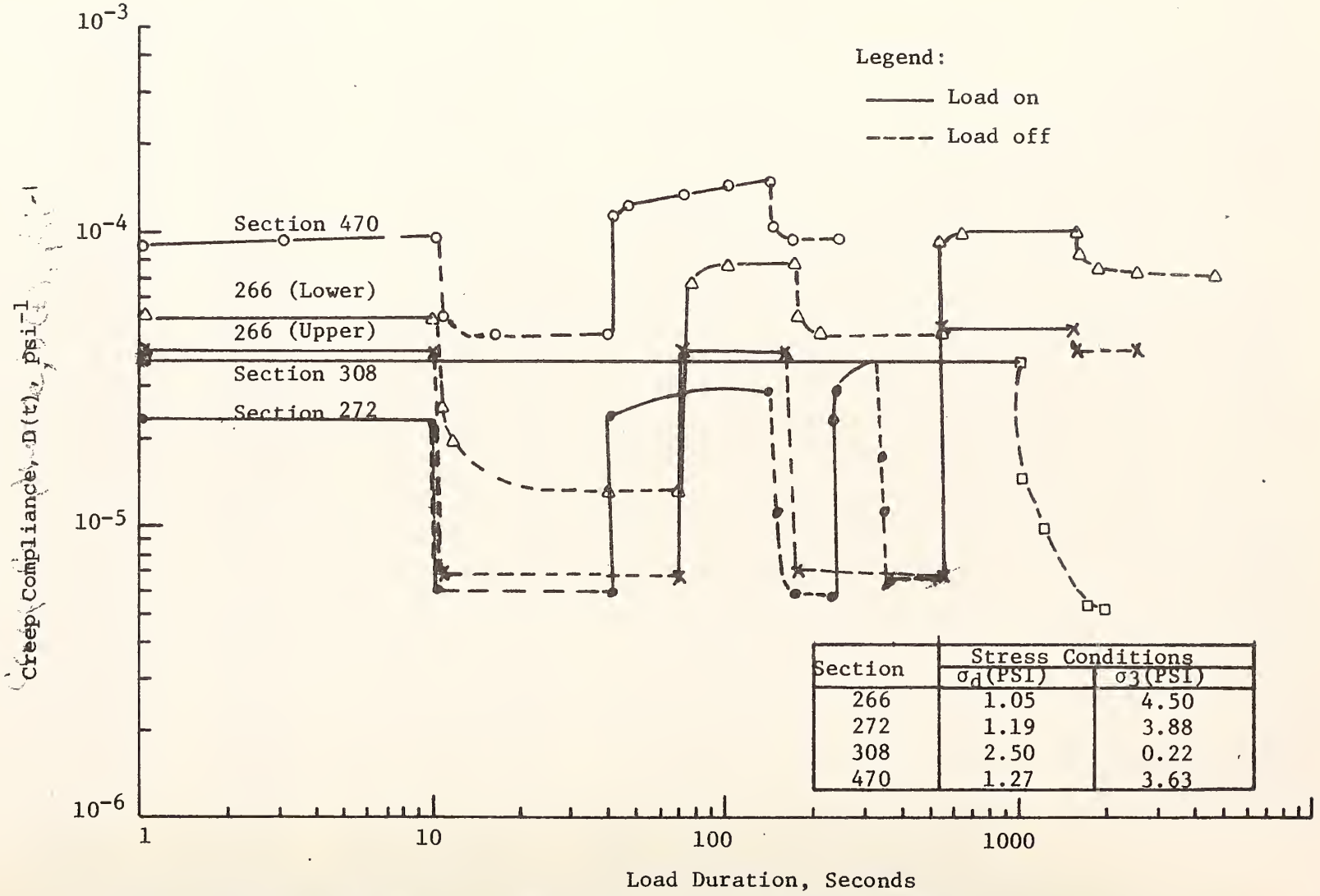

Figure 43. Creep compliance test results (tested after dynamic conditioning) for AASHO Road test subgrade undisturbed samples.

The values of creep compliance used were:

Section No.	Creep Compliance, PSI x 10^{-5}
266	4.50
272	2.35
308	3.75
470	9.00

The value for section 266 is an average of the values obtained for the upper and lower samples.

Stochastic Inputs

The magnitudes to use in the sensitivity analysis for coefficients of variation VARCOEF1, VARCOEF2 and VARCOEF3, for LAYER1, LAYER2 and LAYER3 respectively are very difficult to arrive at due to the limited data available.

A low value of 0.10 and a high of 0.30 was selected for VARCOEF1 after study of Reference 12 which included the results of substantial study of variability in modulus of elasticity for asphaltic concrete cores from operating highways. The same values were selected for VARCOEF2 after study of References 12 and 14. It was reasoned in each case that the variability in creep compliance should approximate that for resilient modulus as they are related.

Reference 33[1] provided the most information on variability and its causes for subgrade resilient moduli, which was again considered to reasonably represent variability for creep compliance. The "between-project" variability can be gross for subgrades due to the wide range of soils, moisture conditions, previous stress history and other causes, but the "within-project" variability for a particular section of highway should be much less. It was considered that around .325 might be a reasonable mean value for subgrade coefficient of variation and that a low of 0.25 and a high of 0.40 might represent a reasonable standard deviation around the mean.

As the level of quality control for the AASHO Road Test construction was very good, less variability might be expected. On a judgmental basis, VARCOEF1 and VARCOEF2 were established at 0.18 and VARCOEF3 at 0.25 for prediction of AASHO Road Test pavement performance.

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[1]Darter, M. I., "Variability Associated with the Resilient Modulus of Subgrades," Report to the Department of Civil Engineering, University of Texas, November 29, 1971.

APPENDIX C

RATIONAL VALUES FOR PERMANENT DEFORMATION PARAMETERS ALPHA AND GNU

The procedures for calculating permanent deformations at the pavement surface used in VESYS IIM are described in Chapter II, including the derivation of the terms ALPHA and GNU (also called α and μ) to simplify the mathematical expression. As these terms are new to the engineering profession, only a few tests had been run specifically to evaluate ALPHA and GNU. To obtain reliable magnitudes of ALPHA(1) and GNU(1) for the asphaltic concrete surface layer, ALPHA(2) and GNU(2) for the base and ALPHA(3) and GNU(3) for the subgrade, it was necessary to search the literature for long term dynamic tests and to run other tests in conjunction with characterizing AASHO Road Test materials.

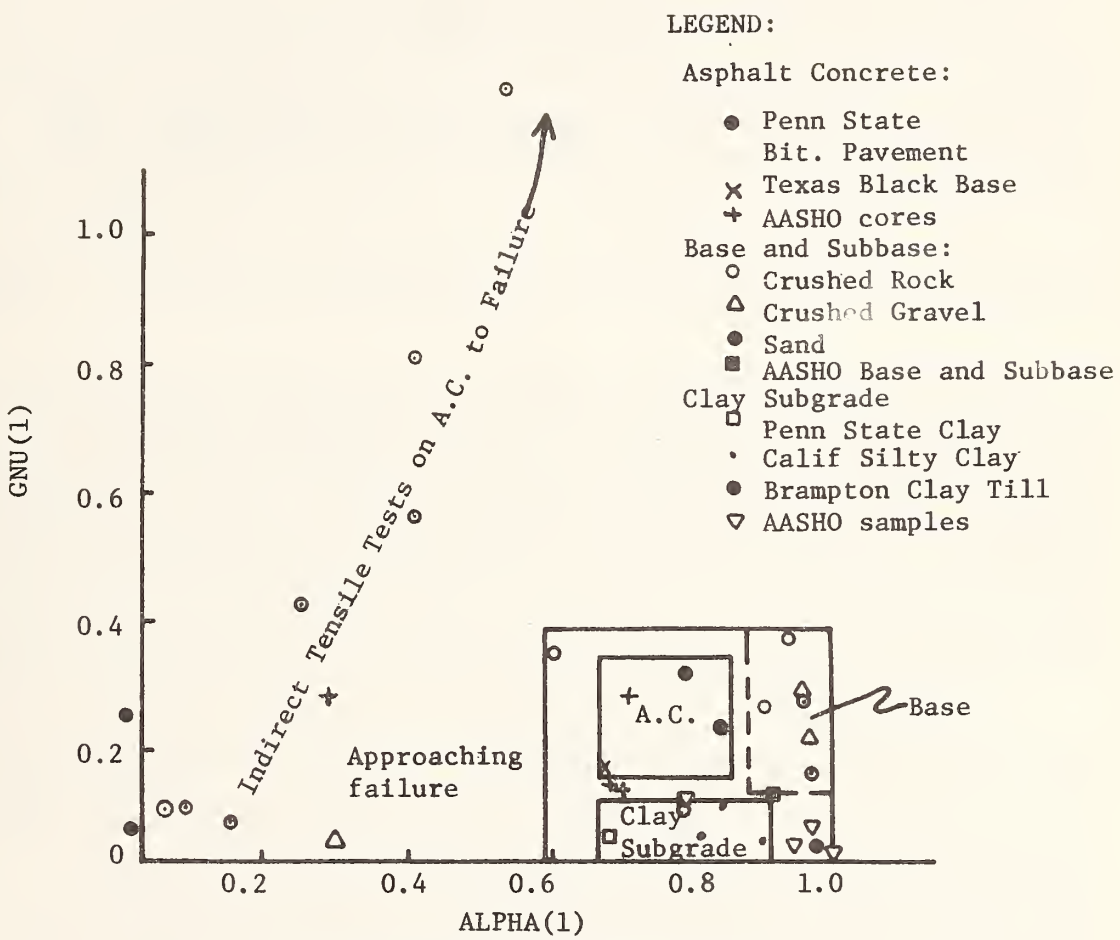
As a beginning toward understanding the nature of ALPHA and GNU, the limited data available were plotted as shown in Figure 44. As can be seen, granular materials have an ALPHA approaching 1.0 and considerable variation in GNU. As ALPHA is one minus the slope of the line drawn through data points as shown in Figure 4, Chapter II, this means that the slope of the line is very small or that the permanent deformation during a load cycle is very small. This is consistent with the known virtually elastic response of well-compacted granular materials. A range for ALPHA of about 0.83 to 0.96 appears reasonable.

Clays and asphaltic concretes display more creep under load and thus more slope on the log-log plot of permanent strain versus number of load applications. Thus ALPHA was lower valued in a range from around .63 to .87 for clay and apparently in a similar range for A.C. unless failure is approaching (discussed in more detail subsequently).

GNU is a much more difficult parameter to attach physical significance to as it is directly dependent on the slope and intercept of the line drawn on the log-log plot and on the inverse of the strain. It appears to be sensitive to the density of the material and the magnitudes of the permanent strains early in the dynamic test as well as the elastic and permanent strains. However, it appears that GNU(3) will generally be less than 0.1 and GNU(2) less than 0.4 with possible excursions to much higher values as seen from Figure 44. GNU(1) is quite variable and may be as high as 1.5, 2.0 or even more (as will be discussed later).

Asphaltic Concrete Surface Layer

Considerable additional information became available from several sources on relatively long-term dynamic tests on A. C. materials



Other Values Off Scale of Plot

Data Source	ALPHA(1)	GNU(1)
AASHO A.C. Core Test	0.83	1.76
AASHO Base	0.94	2.68
	0.92	2.62
AASHO Subbase	0.97	1.99
AASHO Subgrade	0.99	3.49

Figure 44. Plot of GNU(1) versus ALPHA(1) for various materials

subsequent to the initial study reflected in Figure 44. However, in some respects the continued study raised as many questions as were answered. For instance, ALPHA(1) and GNU(1) unlike the elastic modulus are considerably dependent on stress level. What stress level should be used in testing? ALPHA(1) and GNU(1) are temperature-dependent, but are not used in VESYS II (M) as temperature-dependent. At what temperature should the test be run?

Even after these questions were dealt with to the extent possible, calculated rutting was still generally high and additional analysis was required to find out why and to further refine the magnitudes of ALPHA(1) and GNU(1) for use in the sensitivity analysis. These studies and the values selected are discussed in detail in the two following subsections.

The Nature of ALPHA(1) and GNU(1) and Stress Level for Tests to Evaluate Their Magnitudes

One of the most difficult aspects of using any sort of pavement model for predicting responses to a set of independent variables is the selection of test conditions for material characterizations. The elastic moduli of the materials in the various layers are dependent on such things as stress state, density, moisture content, temperature, duration of loading, frequency of loading, etc. The relation between permanent deformation and number of load cycles is dependent on much the same variables but not according to the same rules. For instance, the dynamic modulus of asphaltic concrete is only mildly affected by stress level while the permanent deformation coefficients are directly affected.

One of the most important decisions for testing to arrive at the parameters ALPHA(1) and GNU(1) from a dynamic test is what stress level to use. The approach commonly used is to estimate the stiffness and poisson's ratios for the various layers and to run elastic layer solutions to predict the average states of stress and strain in each layer (assumed to occur at mid depth for the base and subbase and at some appropriate depth into the subgrade). As the moduli themselves for the base, subbase, and subgrade materials are stress-dependent, some iteration may be necessary. For Task D of this project (comparisons between predicted and measured responses), a three-level range of elastic moduli were run for the subgrade and these elastic moduli and estimated resilient moduli were plotted versus deviator stress from the solutions. The intersection of these relations represented the best estimate for a testing deviator stress. This worked very well for a selected depth (18 inches in this case) within the subgrade as the resulting resilient modulus from testing reasonably approximated that obtained from the intersecting curves discussed above.

Stress Distributions in Asphaltic Concrete Surface Layers -The problem is even more complex for the surface layer as relatively high tensile

stresses may occur near the bottom of the layer, which apparently contributes the most to rutting based on the Brampton Road Test ana other observations. Figure 45 shows the calculated stress distributions in the twelve-inch layer (original and overlay) for section 470 of the AASHO Road Test pavements calculated by elastic layer theory for dual wheel loads of 4000 lbs. each and with a tire pressure of 75 psi. Note that the deviator stress and the vertical stress have their maximum at opposite locations and that the deviator stress near the bottom is primarily composed of tension in the lateral direction.

Figure 46 provides the calculated stress distributions for a thin 1½ inch thick asphaltic concrete layer. In this case, the vertical compressive stresses are not reduced greatly by depth and the tensile stresses are much greater, culminating in a deviator stress ranging from zero at the surface to around 139 psi at the bottom. Figure 47 gives estimated stress distributions for an intermediate thickness of six inches.

ALPHA(1) and GNU(1) as Functions of Deviator Stress in Asphaltic Concrete - ALPHA(1) and GNU(1) are input for VESYS IIM as if they were invariants, but they are actually variables themselves with stress, temperature, mix, etc. While the mix can be reasonably represented in sample preparations, the average stresses in the layer must be estimated and some selected temperature used.

It will be useful to see how ALPHA(1) and GNU(1) vary with deviator stress and temperatures. Fortunately, sufficient observations exist for an indicative comparison. Table 12 includes the axial strains from a factorial of repetitive-loading triaxial tests run on the asphaltic concrete base materials used in the Brampton Road Test. These were run at the University of Waterloo on identical materials (to the extent possible) while varying only vertical stress, lateral stress and temperature of the sample (Ref 16). Also included in Table 12 are identification of the test temperatures and stresses and values of ALPHA(1) and GNU(1) derived from Figure 48. All tests were to 700,000 cycles of load. Similar information is provided in Table 13 for testing by Brown and Snaith (Ref 17).[1]

As Dr. Morris did not measure resilient strains, the values from Asphalt Institute tests on the same materials were used. The densities of the Asphalt Institute specimens were about 4 PCF higher, so the resilient strains should be smaller. This probably means that the values of GNU(1) calculated are slightly high.

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[1] Brown, S. F. and M. S. Snaith, "The Permanent Deformation Characteristics of a Dense Bitumen Macadam Subjected to Repeated Loading", Proceedings, Association of Asphalt Paving Technologists, Volume 43, 1974.

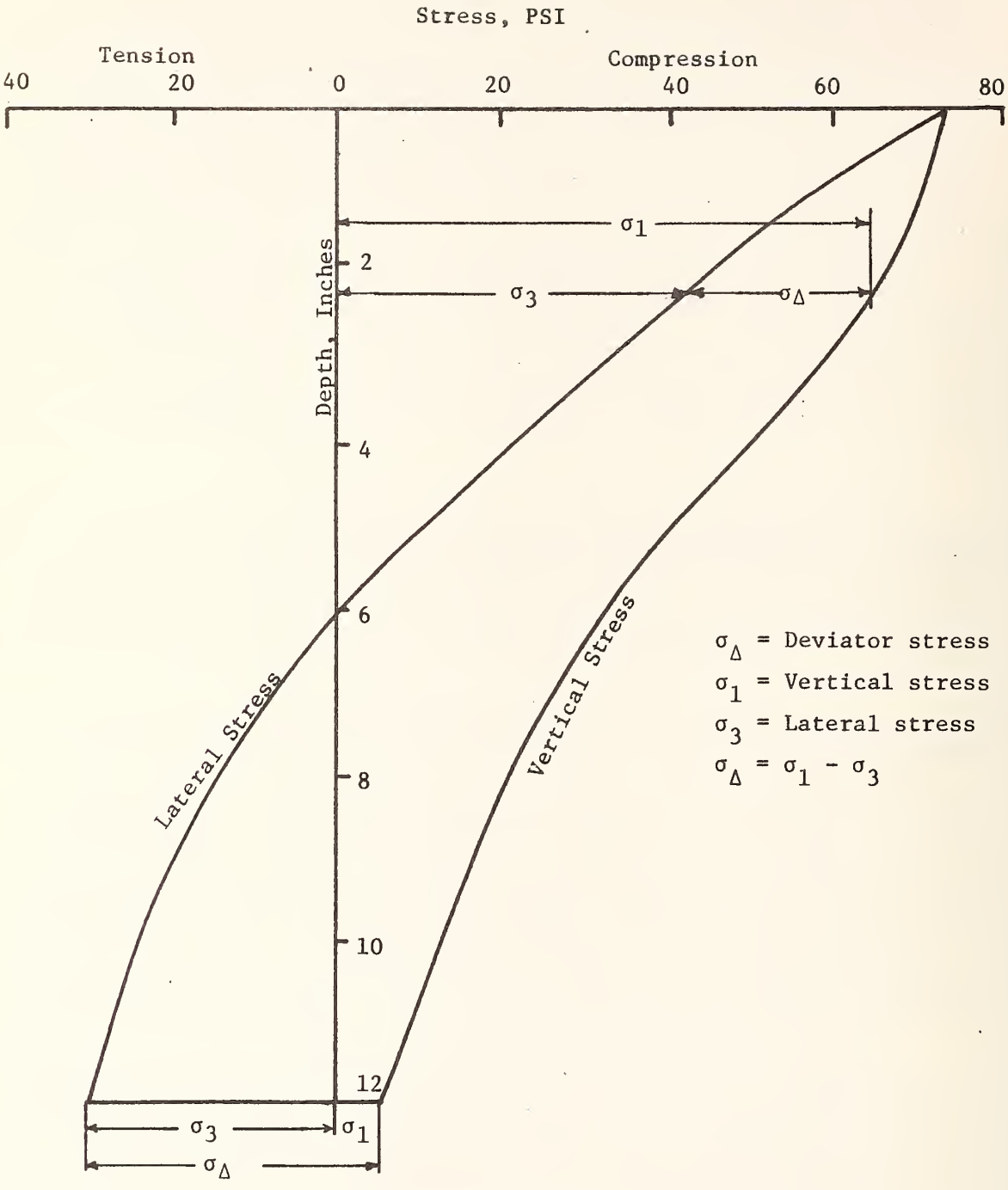


Figure 45. Calculated stress distributions for section 470 of AASHO Road Test

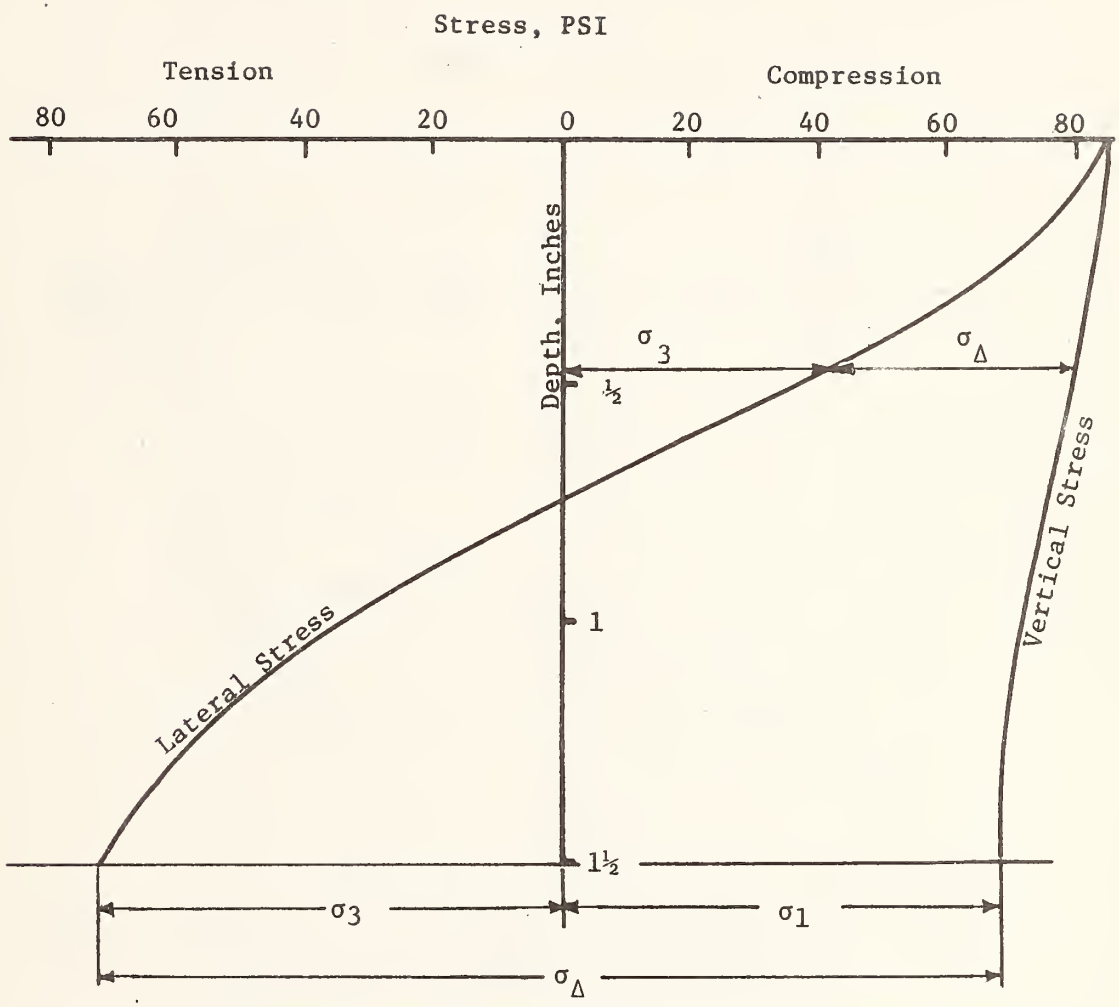


Figure 46. Calculated stress distributions for a thin asphalt concrete layer typical for subdivision streets.

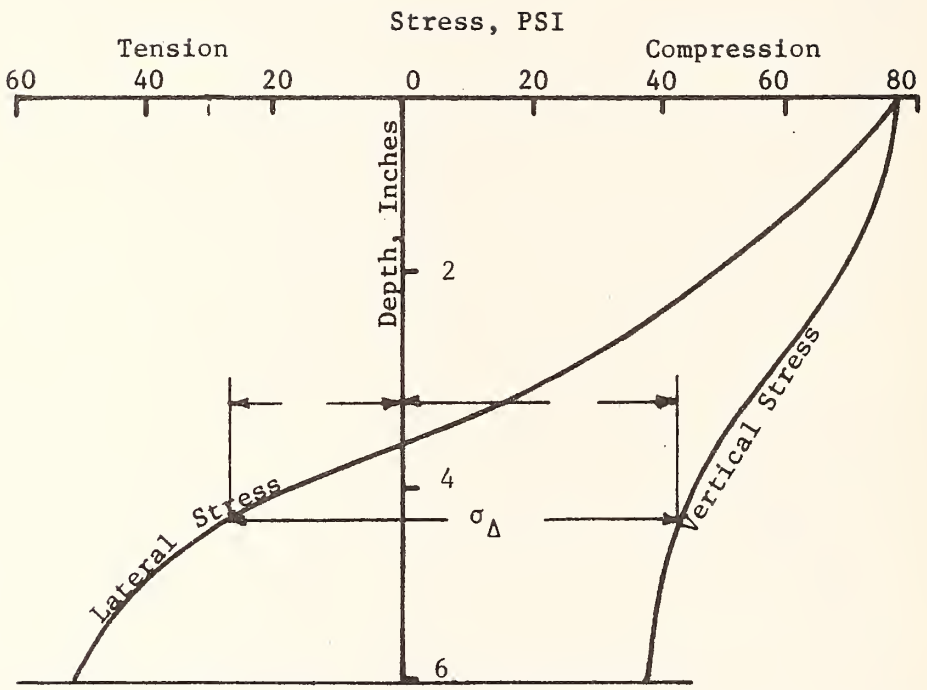


Figure 47. Estimated distribution of stresses for a 6-inch A.C. layer arrived at by a judgmental interpolation between values on Figs. 45 and 46.

Table 12 Measured axial strain compression series - Brampton asphaltic concrete base (After Dr. Jack Morris, Sept. 1973)

	TEST NUMBER	SAMPLE NUMBER	CONDITION*	AXIAL STRAIN % N x 10^6 1	3	10	30	70	Approx. Dynamic Modulus E* psi	Approx. Resilient Strain $\Delta\varepsilon$ x 10^{-4} in/in	I	S	GNU(1)= $I3/\Delta\varepsilon$	ALPHA(1)= 1-S	Temp. ^oF	σ_1 Psi	σ_3 PSI
MAIN FACTORIAL	1	20	● ac	1.30	2.35	4.97	7.72	9.90	120,000	3.75	.00054	.39	.56	.61	90	65	20
	2	02	b	0	0	0	0	0									
	3	38	c	0.71	1.69	3.33	4.83	6.00									
Tensile	4	40	abc	0.12	0.17	0.21	0.26	0.29	120,000	1.25	.00049	.1316	.52	.87	90	65	80
	5	01	Δ a	0.36	0.77	1.43	2.40	3.05	580,000	.776	.0001	.43	.55	.57	65	65	20
	6	23	⊕ a	0.56	0.82	1.12	1.39	1.59	580,000	.776	.0019	.1564	3.83	.84	65	65	20
	7	27	(1)	0.11	0.16	0.22	0.28	0.32			error noted 6, May 1975						
Tensile	8	26	ab	0.16	0.23	0.32	0.40	0.47	580,000	.259	.0003	.204	2.36	.80	65	65	80
	9	42	b	0.02	0.04	0.06	0.07	0.08									
	10	06	bc	-0.11	-0.18	-0.24	-0.30	-0.37									
	11	09	x c	1.45	2.13	2.89	3.59	4.11	120,000	2.08	.0030	.19	2.74	.81	90	45	20
	12	17	(1)	0.18	0.27	0.36	0.45	0.52									
Tensile	13	08	abc	0.13	0.20	0.28	0.35	0.40	120,000	1.25	.00052	.15	.62	.85	90	65	80
	14	14	■ ac	1.30	3.15	6.68	10.00	12.60	120,000	3.75	.00195	.30	1.56	.70	90	65	20
	15	44	bc	-0.24	-0.38	-0.50	-0.64	-0.73									
Tensile	16	12	ab	0.05	0.07	0.10	0.12	0.14	580,000	.259	.00012	.19	8.82	.81	65	65	80
CENTRE AND STAR POINTS	17	34	o_{10}	0.22	0.32	0.43	0.52	0.60									
	18	30	Centre	0.16	0.21	0.30	0.35	0.40	270,000	.185	.00052	.15	4.05	.85	77.5	55	50
	19	33	Centre	0.05	0.06	0.08	0.10	0.11	270,000	.185	.00026	.12	1.69	.88	77.5	55	50
	20	36	T_0	0.18	0.25	0.33	0.40	0.47									
	21	18	o_{11}	0.15	0.25	0.33	0.40	0.45									
	22	21	Centre	0.11	0.16	0.22	0.28	0.31	270,000	.185	.00045	.13	3.16	.87	77.5	55	50
Tensile	23	03	o_{31}	0.07	0.10	0.12	0.16	0.19			.00021	.16		.84	77.5	55	86.4
	24	41	T_1	0.08	0.10	0.13	0.18	0.20									
	25	07	▼ o_{30}	0.11	0.17	0.23	0.28	0.30	270,000	1.53	.00074	.105	.51	.90	77.5	55	13.6

* Code, o_1 =a; o_3 =b; T = c
Inclusion of code letter indicates that factor is at its high level.
Suffix 1 = High level star points; 0 = Low level star points.

Note: E^* and $\Delta\varepsilon$ based on data for asphaltic base tests by Asphalt Institute with Mean density some 4 pcf higher than these test samples. $\Delta\varepsilon$ probably would really be larger and μ smaller.

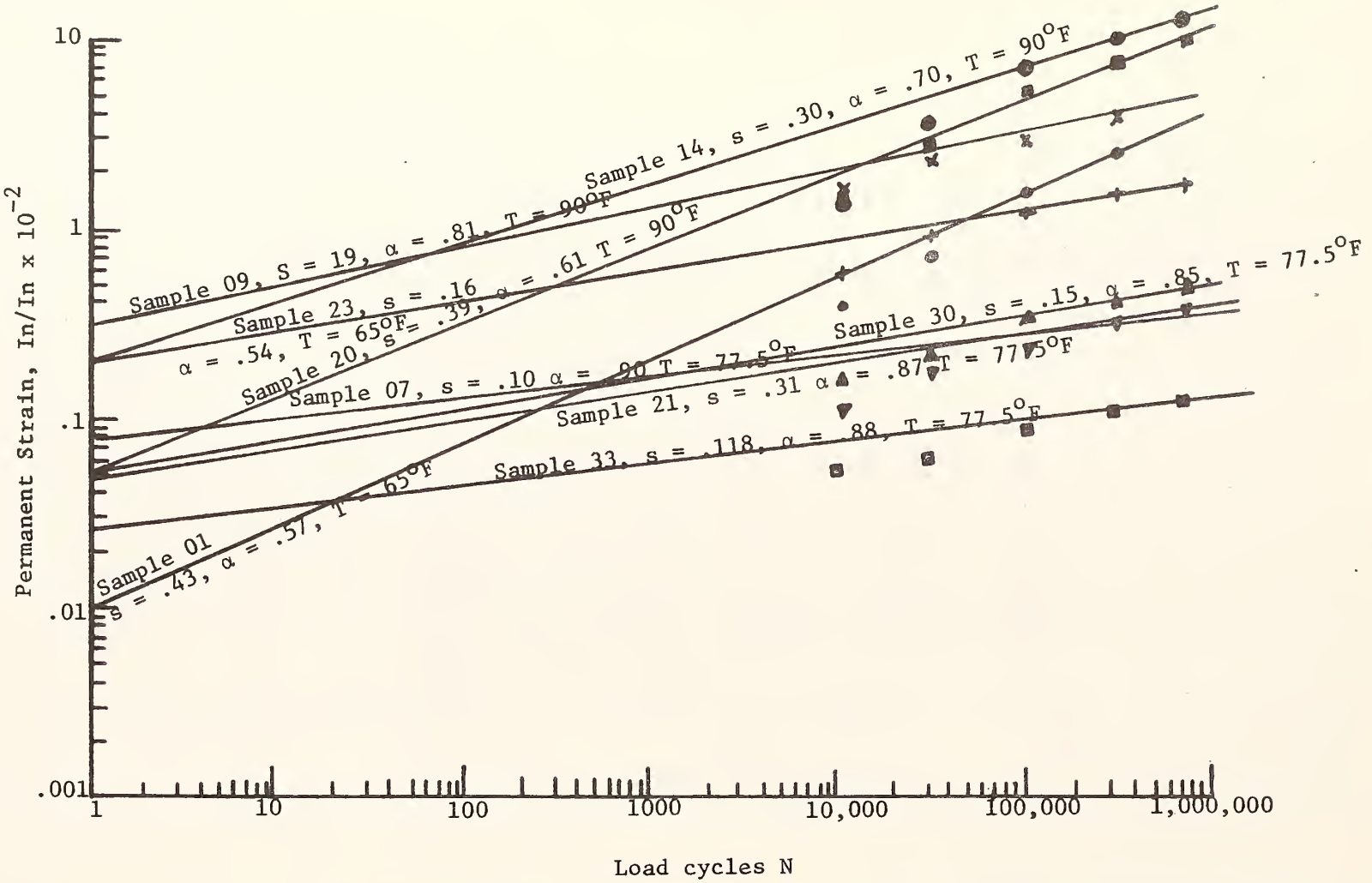

Permanent Strain, In/In x 10^{-2}
10
1
.1
.01
.001
1
10
100
1000
10,000
100,000
1,000,000
Load cycles N
Sample 09, S = 19, α = .81, T = 90°F
Sample 14, s = .30, α = .70, T = 90°F
Sample 23, s = .16
α = .54, T = 65°F
Sample 20, s = .39, α = .61 T = 90°F
Sample 07, s = .10 α = .90 T = 77.5°F
Sample 30, s = .15, α = .85, T = 77.5°F
Sample 21, s = .31 α = .87 T = 77.5°F
Sample 33, s = .118, α = .88, T = 77.5°F
Sample 01
s = .43, α = .57, T = 65°F

Figure 48. Plots of permanent strain versus load cycles for Brampton asphaltic concrete base (after Morris, Ref 49).

Table 1 3 Permanent Deformation Data from Brown and Snaith Paper

*** Fig. No.	Vertical Stress (PSI)	Lateral Stress (PSI)	Deviator Stress (PSI)	Temp. (°F)	Vertical Strain x 10^{-2} N x 10^3 (see note I) .1	1	Cycles / ε	10	Cycles / ε	100	Cycles / ε	** Est. (in $\frac{\Delta\varepsilon}{\text{in x } 10^{-4}}$)	I ($\frac{\text{in}}{\text{in x } 10^{-4}}$)	S	GNU(1) $\frac{IS}{\Delta\varepsilon}$	ALPHA(1) 1-S
5	21.7	0	21.7	68	.120	.244		.383	47 / .5	.549	480 / .739	1.03	13	.14	1.77	.86
	58	0	58	68	.182	.338	4 / .5	.634	39 / 1.0		82 / 1.37	2.76	6.4	.25	.58	.75
*	94	0	94	68	.253	.634	3.4 / 1.0	1.80	7.8 / 1.5			4.48	2.9	.44	.28	.56
	130	0	130	68	.295	1.11	2.9 / 2.0		.42 / .75			6.19	2.2	.56	.09	.44
6	94	0	94	50	.093	.229		.479		.983	280 / 1.55	2.16	2.6	.32	.39	.68
	94	0	94	86	1.16		.16 / 1.5		.25 / 2.0			8.10	5.8	.63	.45	.37
* 7	94	0	94	68								4.48				
	94	14.5	79.5	68	.329	.714		1.17	17 / 1.50			3.79	17.5	.20	.92	.80
	94	29	64	68	.279	.557		.893		1.26		3.05	20	.16	1.05	.84
8c	94	35 psi const 10.5 dynamic	80	68	.343	.757		1.80		3.57		3.79	10	.31	.82	.69
8d	94	14 const.	80	68	.257	.686		1.36		2.11		3.79	22	.20	1.16	.80

Note 1 - 3 columns left for choice of number of cycles for a data point.

*Apparently some variation under identical stress and temperature conditions
**Dynamic modulus taken approximation from Fig. 18 as 210,000 psi for 68°F, 435,000 psi for 50°F and 116,000 at 86°F.
***Figure numbers are from Brown and Snaith Report, Ref. 51.

Note first on Fig. 48 that samples 21, 30, and 33, which represent high stresses but low stress difference, plot at very low slopes relative to other samples with higher deviator stresses. This is again illustrated by Samples 09 and 14, whose test conditions were identical except that Sample 09 had a lesser vertical stress and therefore less deviator stress. This means that ALPHA(1) should be larger for the material near the surface of the layer where such stresses occur (See Fig. 45).

Figures 49 and 50 show plots attempting to find a correlation between ALPHA(1) and GNU(1) and deviator stress. These plots include the Brampton data mentioned above, the Brown and Snaith data, McLean and Monismith State Test Track, a Texas "black base" specimen, cores recently taken from sections of the AASHO Road Test pavements after many years of use and cores from Texas highways tested to fatigue failure in dynamic indirect tension. As can be seen, both ALPHA(1) and GNU(1) appear to be strongly correlated to deviator stress. The hatched bands indicate the approximate regions that points occurred in the graphs, except that Brown and Snaith's values for ALPHA(1) were much higher than the American and Canadian mixes of higher asphalt content. A line has been drawn for this data exclusive of the hatched area.

As can be seen, both ALPHA(1) and GNU(1) decrease with increasing deviator stress, but at different rates.

It is of interest now to see how ALPHA(1) and GNU(1) might vary with depth in asphaltic concrete surface layers. Using the deviator stresses plotted in Figures 45, 46, and 47 and the values of ALPHA(1) and GNU(1) from the hatched areas of Figures 49 and 50. Figure 51 has been plotted to show the resultant distributions of ALPHA(1) and GNU(1),

ALPHA(1) varies across the range for the thin section, nearly across for the 6-inch thickness and only in a narrow band for the thick section. It would apparently be desirable to select a stress level for testing that would provide an average value of ALPHA for use by the computer program across the entire depth. This appears to have occurred at about 3½ and 6½ inches, respectively, for the sections in order of increasing depth. This depth ranges from .50 to .58 of the layer thickness.

Perhaps a reasonable rule would be to use the deviator stress estimated through use of elastic layer theory at a depth of .55 of the layer thickness for testing. The resulting values of ALPHA(1) would apparently represent as good an average value as we can obtain.

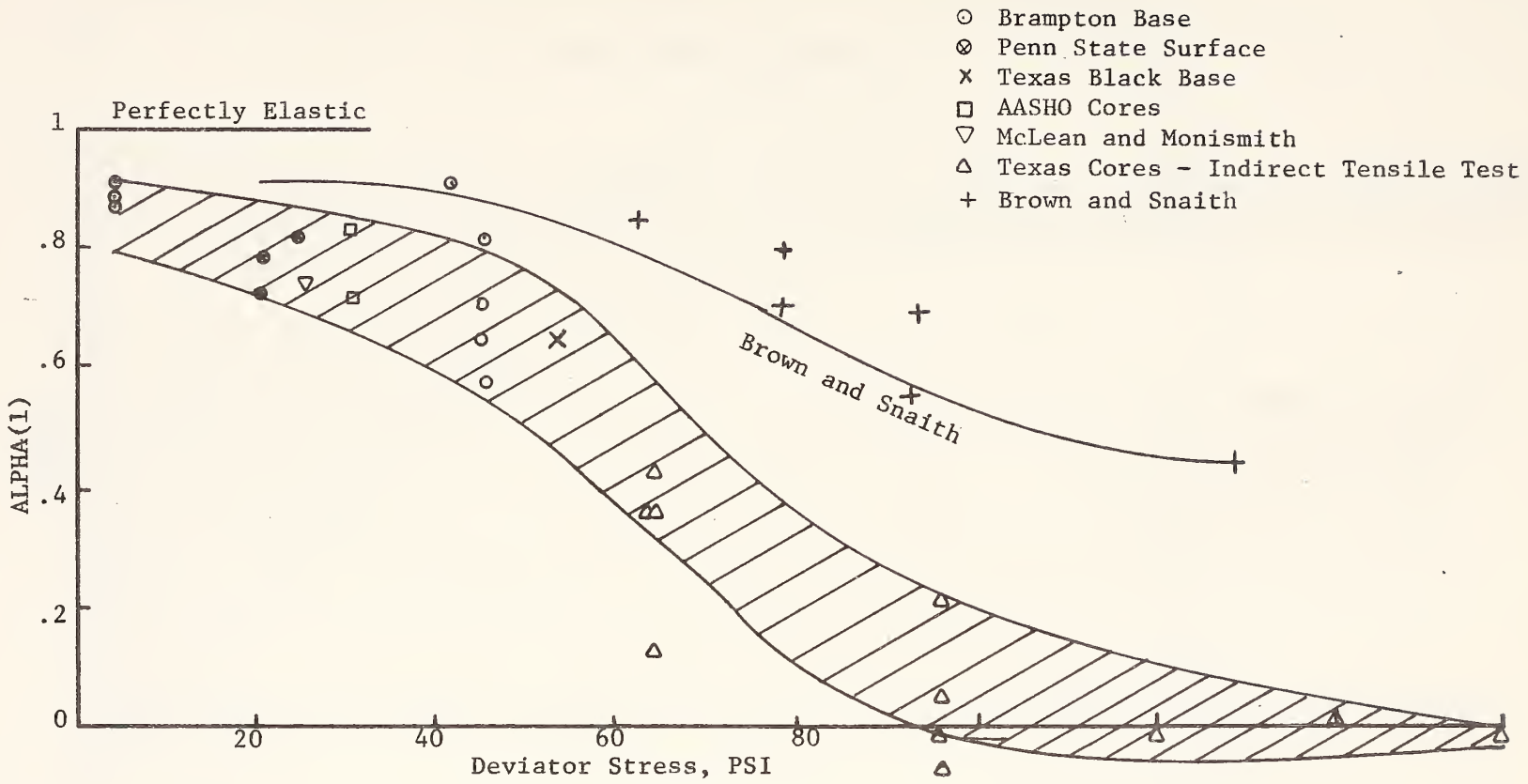


Figure 49. Plot of ALPHA(1) versus deviator stress for asphaltic concrete

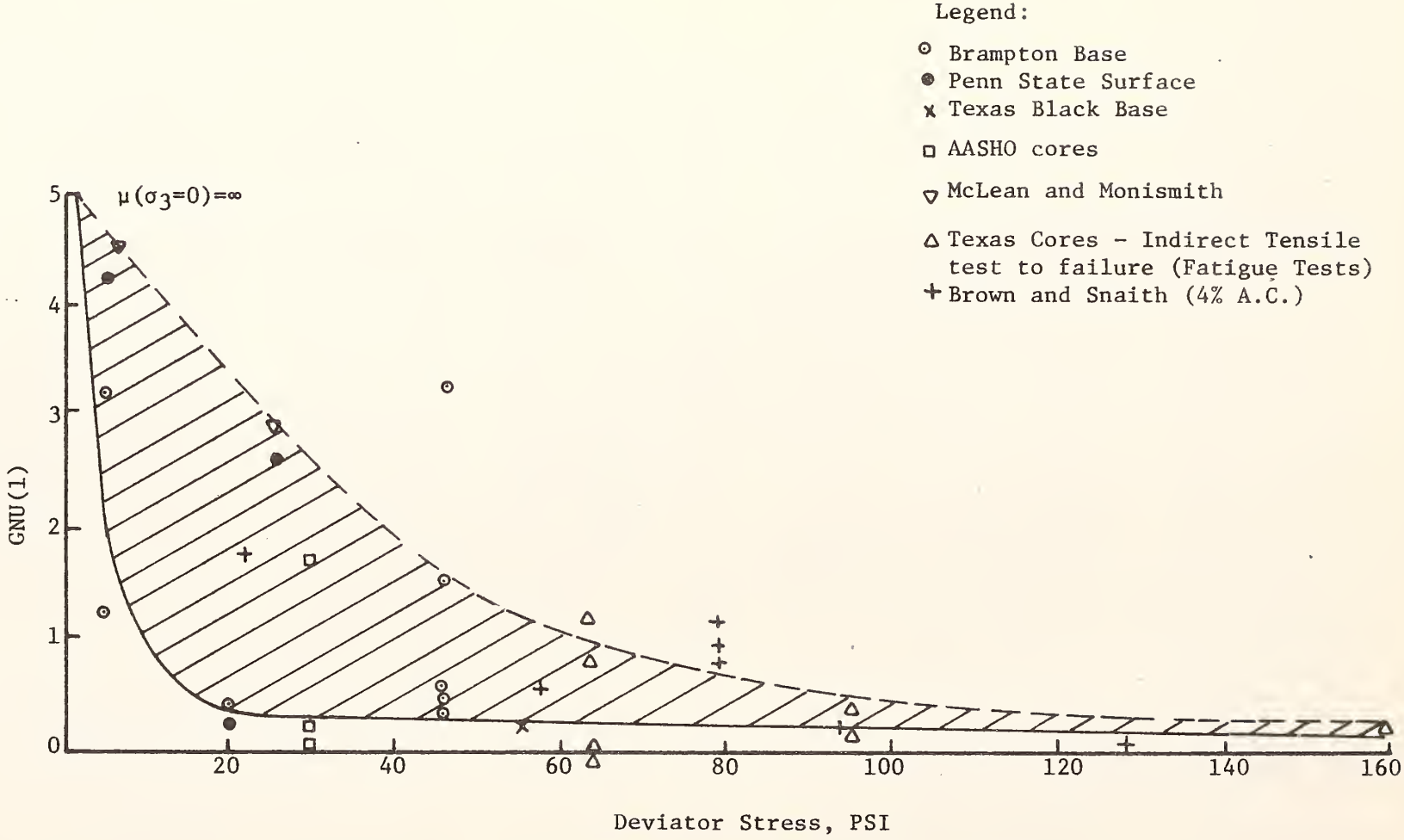


Figure 50. Plot of GNU(1) versus deviator stress for asphaltic concrete.

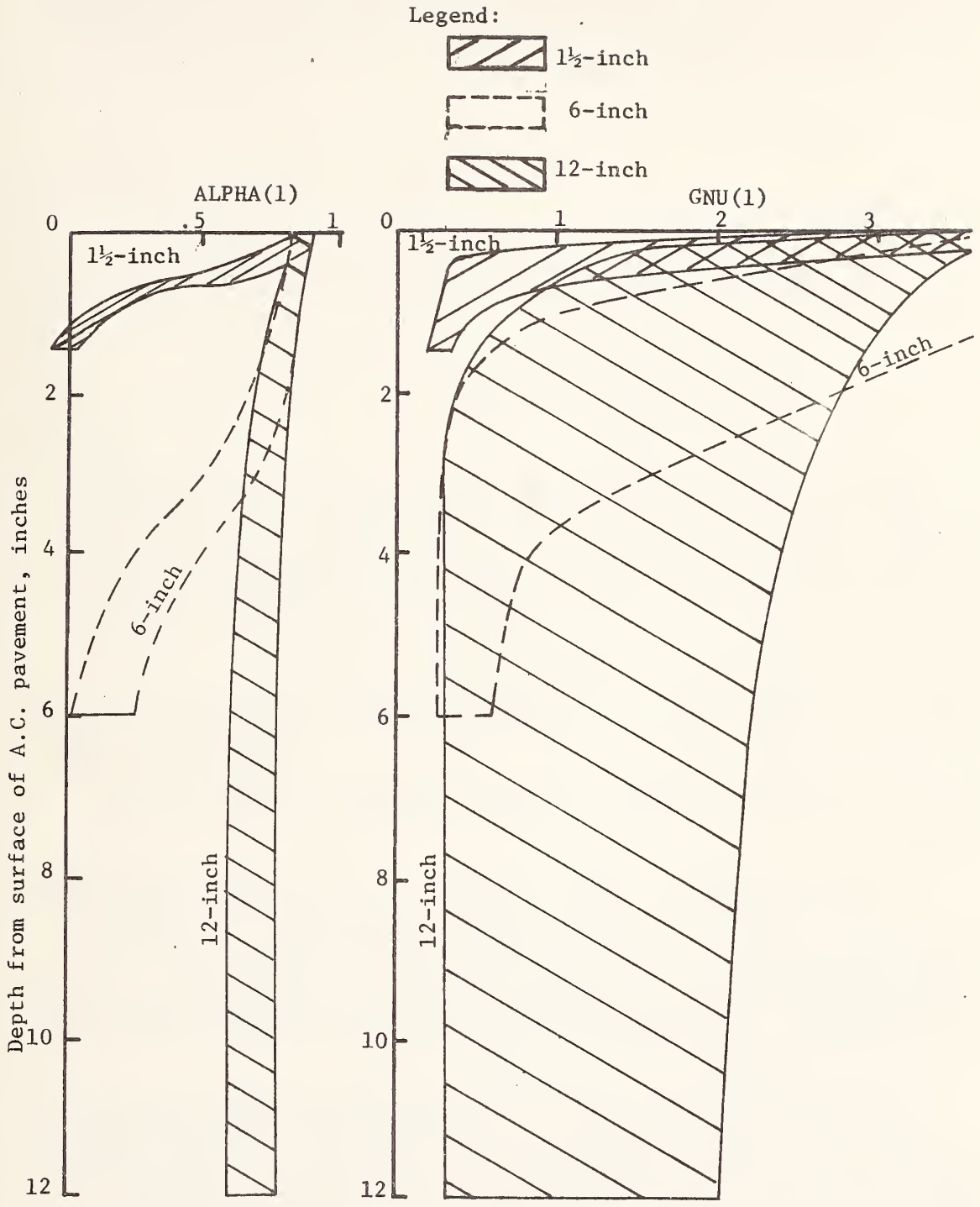


Figure 51. Distribution of ALPHA(1) and GNU(1) with depth in 1½-inch and 12-inch layers of asphaltic concrete.

As can be seen, the testing stress proposed above does not apparently provide average values for GNU(1), but this may not be important for the reasons discussed below.

The equation for GNU(1) is:

$$GNU = \frac{IS}{\Delta\varepsilon}$$

Where

I = strain intercept (not a logarithm) on a log-log plot of accumulated permanent strain versus number of load cycles (See Figure 4).

S = Slope of the straight line drawn through the points representing the higher number of load cycles on the log-log plot.

$\Delta\varepsilon$ = resilient strain

As the resilient strain is essentially constant and the slope (1-ALPHA) does not vary greatly at a particular stress level (see Figure 49, the very large variations in GNU(1) shown in Figure 50 for deviator stresses below about 70 psi (and consequently on Figure 51 for portions of the layers having deviator stresses this low) are primarily due to the strain intercept I. The intercept, which represents the curve-fitted estimate of the permanent strain during the first load cycle is, for a particular stress level, dependent on stress history (or conditioning) and temperature. For instance, sample 01 depicted in Figure 4 indicates a permanent strain of .0001 inch under the first load, .002 inch at 1000 cycles and .0066 inches at 10,000 cycles. For a 6-inch pavement, this would amount to about .04 inch of permanent deformation during the first 10,000 cycles.

If on the other hand, 1000 cycles were applied for "conditioning" as is common in resilient modulus testing, the permanent deformation for cycle 1 (really 1001 counting conditioning) would be .00005 inch, .00082 inch at 1000 cycles and .0045 inches at 10,000 cycles. For a 6-inch pavement, this would amount to .027 inch of permanent deformation or an increase of a predicted .013 inch difference between a value of GNU(1) of .55 and one of about .03.

A similar analysis for sample 14 tested at 90°F however, produced .177 inch of permanent deformation after 10,000 cycles without conditioning and .09 inches with 1000 cycles conditioning as μ varied from around 1.56 to .005. This indicates that the amounts of permanent strain introduced during the first few thousands of load cycles are minor regardless of stress history at low to medium temperatures and stress levels. but may be important at higher temperatures and stress

levels. As the long-term permanent deformations may not be strongly affected; responses calculated by VESYS IIM may be relatively insensitive to GNU and special efforts may not be justified for its accurate determination during testing. If so, it might be omitted as an independent variable in the sensitivity analysis.

ALPHA(1) and GNU(1) as Functions of Temperature in Asphaltic Concrete Surface Layers

Figure 52 presents plots of ALPHA(1) and GNU(1) as functions of temperature derived from Table 12, Figure 48 and other sources. Correlations seem to exist between ALPHA(1), GNU(1) and temperature, but they seem to defer to the effects of deviator stress and approaching failure.

As can be seen, the only serious deviations from the relation plotted for ALPHA(1) are the results from Brown and Snaith and indirect tensile tests on Texas Cores. These were both at relatively high deviator stresses and to failure. The latter means that permanent deformations will at some point increase with number of cycles (see Figure 53), which means a larger slope on the log-log plot and thus smaller values of ALPHA(1). Smaller values of GNU(1) may also be generally expected as the lines will intercept at much lower values of permanent strain for the first cycle.

GNU(1)'s relation is less well defined, but indicates that it will range between .005 and around 1.0 for most specimens with some excursions to higher values, especially for specimens tested at low deviator stresses.

It is indeed surprising to note that ALPHA(1) did not have its maximum at a low temperature and decrease continuously with increasing temperature. There seems to be an established trend for permanent deformations per cycle (low value of ALPHA(1) indicates high slope of line on log-log plot) to decrease from a rather high value at low temperatures to a minimum around 75° and then to increase with temperature from that point. Actually, it is believed that some interaction with deviator stresses or other parameters has caused this apparent phenomenon of high deformations at low temperatures, which is contrary with the large body of knowledge developed on seasonal rutting. For instance, 3 of the 4 points of maximum value were tested at only 5 psi of deviator stress, which wjuld be expected to yield high values of ALPHA(1). It seems more likely that the true relationship for ALPHA(1) at some particular medium-level constant deviator stress might be .85 at 40°F to around .65 at 90°F. At lower deviator stresses, the relation would translate to higher values and to lower values for higher deviator stresses, perhaps as postulated in Fig. 54 on the basis of the American and Canadian data. Fig. 54 indicates expected low-level effects from temperature and higher-level effects from deviator

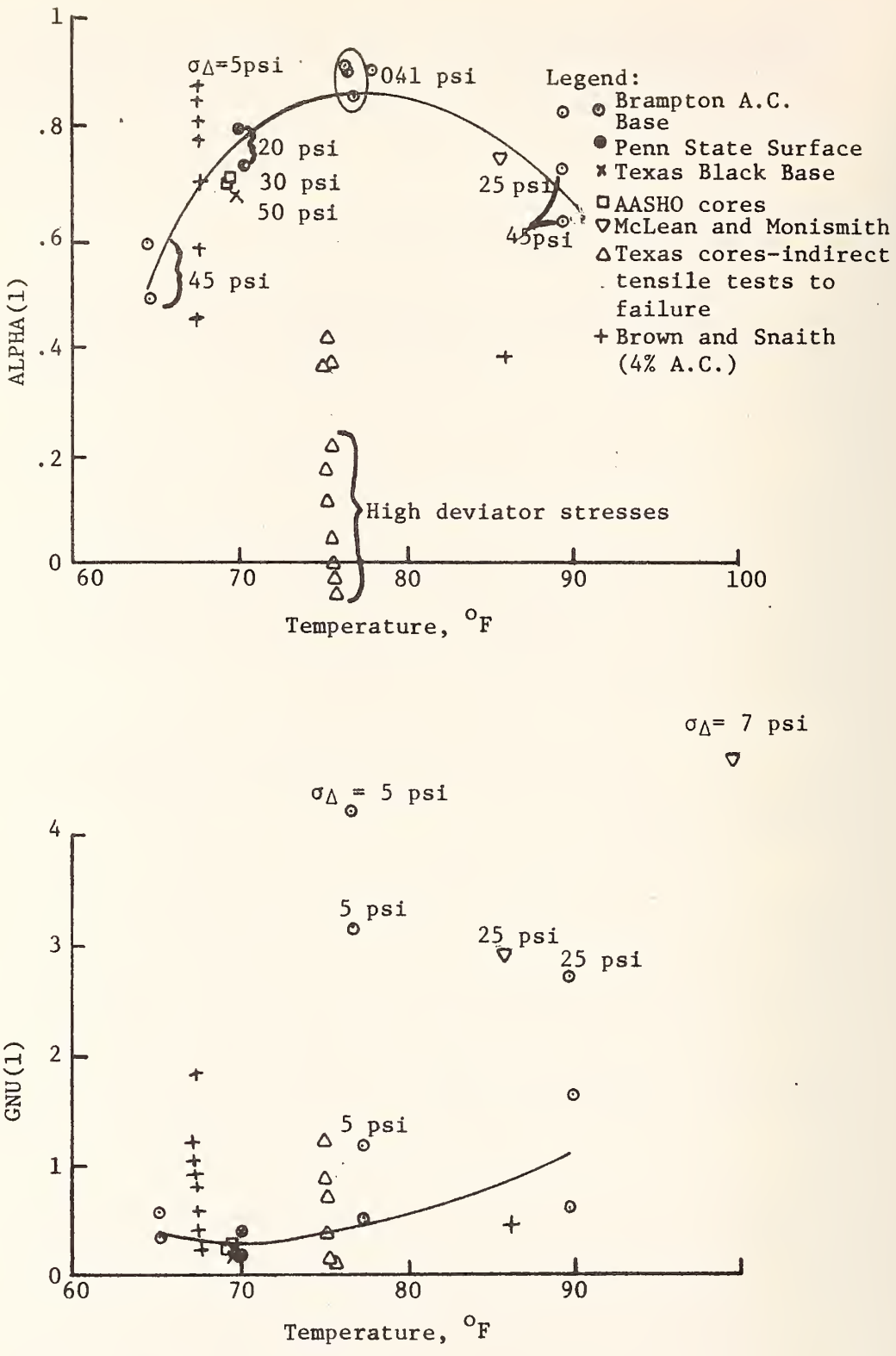


Figure 52. Plots of ALPHA(1) and GNU(1) versus temperature.

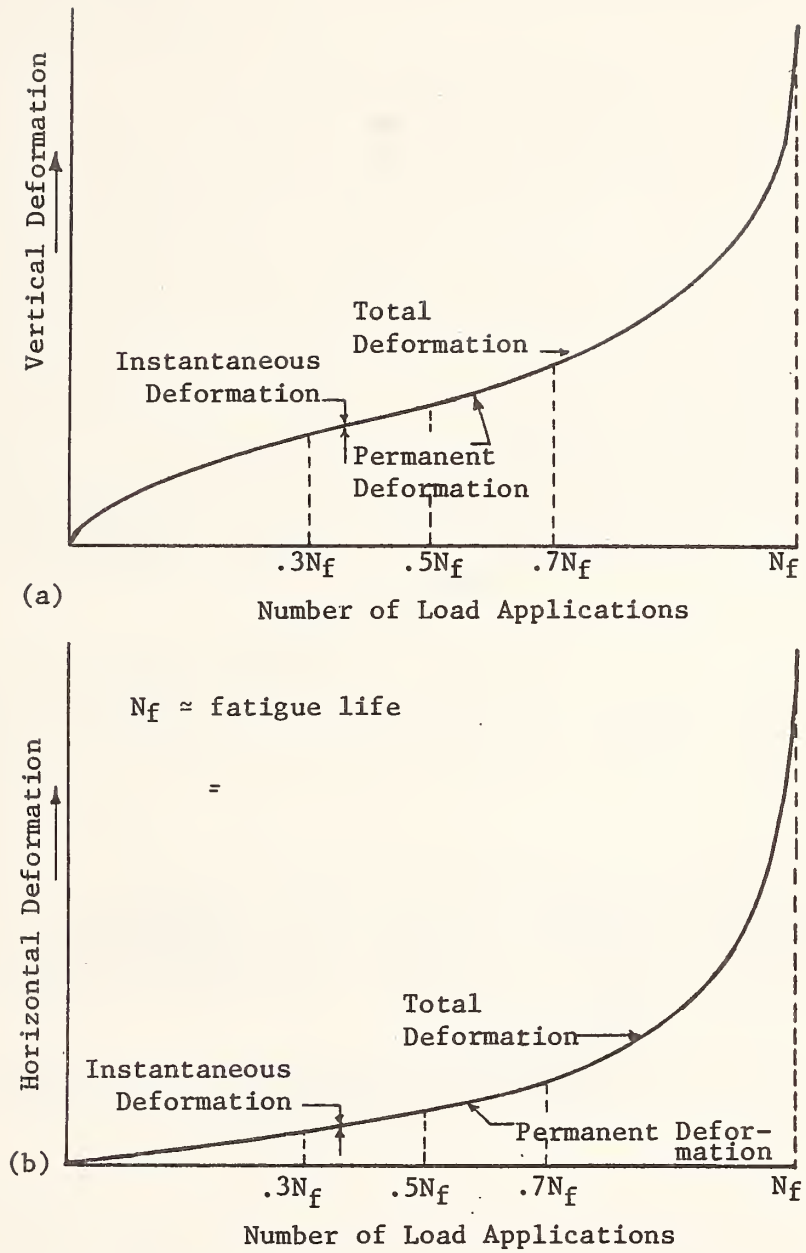


Figure 53. Typical relationship between deformation and number of load applications (After Navarro and Kennedy, Ref. 15).

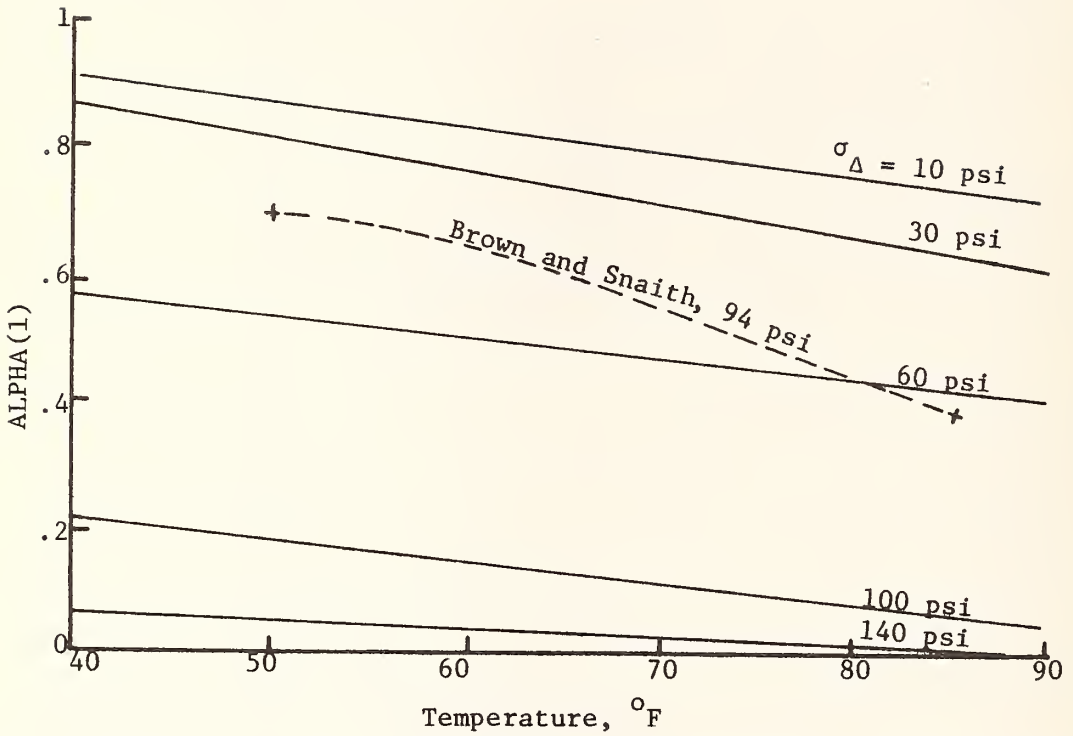


Figure 54. Postulated approximate relationship between ALPHA(1), deviator stress and temperature.

stress. Data was available from Brown and Snaith for one curve included as a dotted line. The general trend is as postulated, but the values of ALPHA(1) are much higher for a particular stress level as indicated in Figure 49.

Temperature should be an important parameter in testing for ALPHA(1) and GNU(1), but it is apparently introduced in VESYS IIM in a different manner. ALPHA(1) and GNU(1) serve to define what fraction of the elastic response to load will remain when the load is removed. The elastic response at the surface is dependent on the inverse of creep compliance values for layer stiffnesses. For the asphaltic concrete layer, a time-temperature shift function makes revisions from the standard 70°F curve to account for actual temperatures. The assumption of VESYS IIM appears to be both that permanent deformations may be calculated on the basis of fractions of elastic deformations and that the effects of varying layer stiffness with temperature will represent the really somewhat different effects of varying permanent deformations with temperature. In a sense, if amounts to applying elastic phenomena to explain plastic phenomena. At any rate, it appears then that the proper test temperature is the one used for the master creep-compliance curve input.

Multiple Regression Results

There were thirty-three "observations" of ALPHA(1) and GNU(1) available from the four sources previously discussed (McLean and Monismith data not available in time for use). This data is listed in Table 14 by source.

Multiple regressions were run with ALPHA(1) and GNU(1) as the dependent variables and deviator stress, temperature and percent asphalt content as independent variables. Analysis of the squares of the mean residuals and F-ratios indicated that the equation below offered the best linear fit:

$$ALPHA(1) = 1.218 - .0047\ \sigma_d - .06A_c$$

$$GNU(1) = 1.985 - .0187\ \sigma_d$$

where

σ_d = Deviator stress in psi.

A_c = Asphalt content in percent

The equation for ALPHA(1) explained 41% of its variance and the equation for GNU(1) explained 48% of its variance. Including temperature and asphalt content only improved the equation for GNU(1) by 1%. The addition of temperature for ALPHA(1) did not improve the percent of variance accounted for at all.

Table 14. Independent and dependent variables for multiple regression

Source of Data	Vertical Stress (psi)	Lateral Stress (psi)	Deviator Stress (psi)	% Asphalt Content	Temperature °F	Measured Responses GNU	ALPHA
Brown and Snaith	22	0	22	4	68	1.77	.86
	58	0	58	4	68	.58	.75
	94	0	94	4	68	.28	.56
	130	0	130	4	68	.09	.44
	94	0	94	4	50	.39	.68
	94	0	94	4	86	.45	.37
	94	0	80	4	68	.92	.80
	94	29	64	4	68	1.05	.84
	94	14	80	4	68	1.16	.80
Brampton 20	65	20	45	5.8	90	.56	.61
01	65	20	45	5.8	65	.55	.57
23	65	20	45	5.8	65	.33	.48
09	45	20	25	5.8	90	2.74	.81
14	65	20	45	5.8	90	1.56	.70
12	65	80	-15	5.8	65	2.94	.81
30	55	50	5	5.8	78	4.05	.85
33	55	50	5	5.8	78	1.69	.88
21	55	55	5	5.8	78	3.16	.87
07	55	14	41	5.8	78	.51	.90
ATE Tests							
266	30	0	30	4.7	70	.16	.67
AASHO 272	30	0	30	4.7	70	1.76	.83
Cores 308	30	0	30	5.1	70	.11	.70
470	30	0	30	4.8	70	.30	.71
U.T. 507L			96	6	75	-.02	.10
Indirect 508L			96	6	75	.05	.12
Tensile 526L			64	6	75	.36	.57
tests 527L			64	6	75	.12	.10

Table 14. Independent and dependent variables for multiple regression (cont.)

Source of Data	Vertical Stress (psi)	Lateral Stress (psi)	Deviator Stress (psi)	% Asphalt Content	Temperature °F	Measured Responses GNU	ALPHA
538L			100	6	75	-.01	.26
403L			96	6	75	.05	.12
458L			64	7	75	.44	1.24
529L			96	6	75	-.08	.07
530L			96	6	75	.22	.41
557L			64	7	75	.36	.83

These equations are plotted in Figures 55 and 56 to show how they compare to the data from which they were developed. As can be seen, these equations are not especially accurate, but are indicative and much better than nothing in the absence of test data. It appears that the dashed curve identified as "alternate curve" in Figure 56 may provide a more meaningful relation for GNU(1) and similar nonlinear curves might improve the fit for ALPHA(1), but there are too many other important variables missing to warrant pursuing a better relation.

Summary on the Nature of ALPHA(1) and GNU(1)

ALPHA(1) is apparently primarily dependent on deviator stress level, and secondarily dependent on temperature, mix, materials, air voids, etc.

GNU(1) is heavily dependent on deviator stress, but is a more complex function of interrelated variables and its relationships are less clear. It appears to range from very small values up to 1.8 in general, with excursions to much higher values for low deviator stresses and other causes. GNU(1) is probably more sensitive to stress history or conditioning than any other variable.

Figure 57 is a plot of all the data available for ALPHA(1) and GNU(1) plotted to relate the two variables. As can be seen, the points fall into specific regions dependent on deviator stress. Again it can be seen that the higher values of GNU(1) generally only occur at low deviator stresses.

As the deviator stress at a depth on the order of .55 of the layer thickness (point recommended previously for selection of test deviator stress) will not typically be below 20 psi, it is believed that the higher values of GNU(1) for lower deviator stresses may be ignored.

The deviator stress selected for testing for a thin pavement might run as high as 90 psi, so ALPHA(1) must be considered across its entire range. This establishes a region defined by ABCDE in Figure 57 with a range in GNU(1) from .01 to 1.8 (zero not allowed) and in ALPHA(1) from 0 to .90 as the region of interest.

It may also be noted to further reduce the region for specific cases that ALPHA(1) < .4 implies pavements having a thickness of less than around 5 inches and that the thicker pavement of say 10 inches or more would have ALPHA(1) > .7.

As ALPHA(1) is heavily dependent on deviator stress and deviator stress is heavily dependent on pavement thickness, it would be possible statistically to develop a range of ALPHA(1) as a function of pavement thickness to be used for insight and as a guide. It would also be

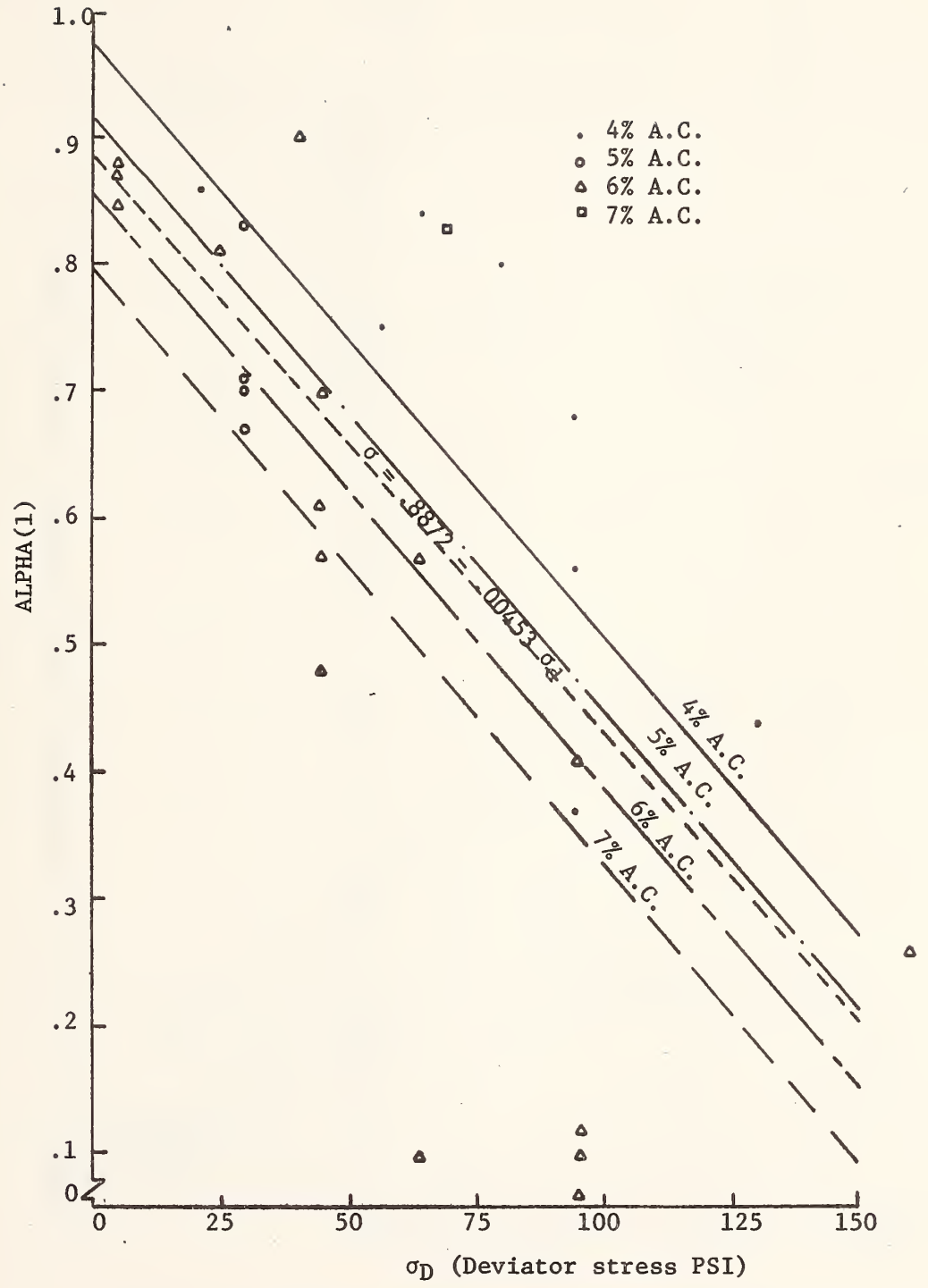
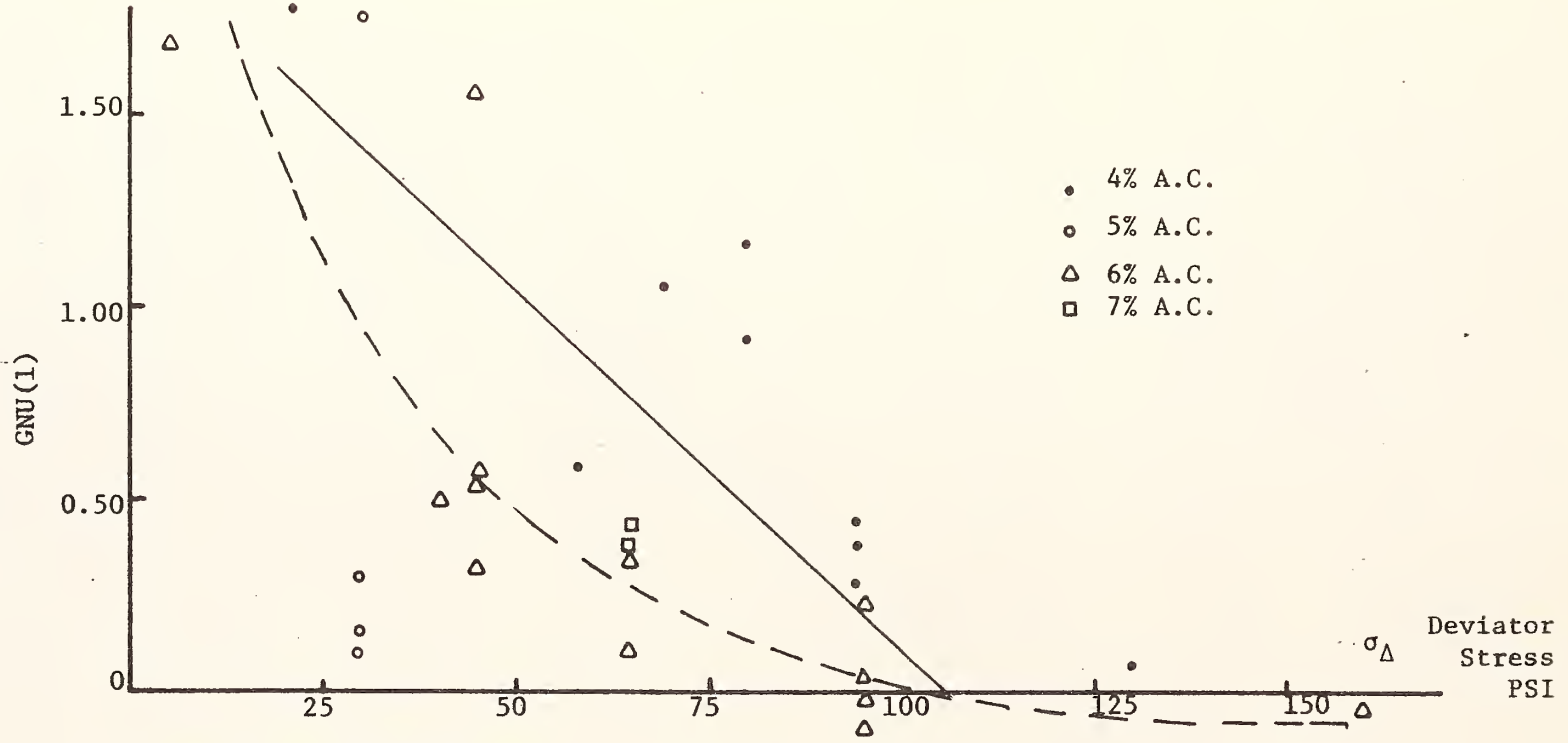


Figure 55. Calculated and measured values of ALPHA(1) versus deviator stress and asphalt content.



Note: Values off Scale

μ	σ_Δ
2.74	25
2.94	-15
4.05	5
3.16	5

Figure 56. Calculated and measured values of GNU(1) deviator stress.

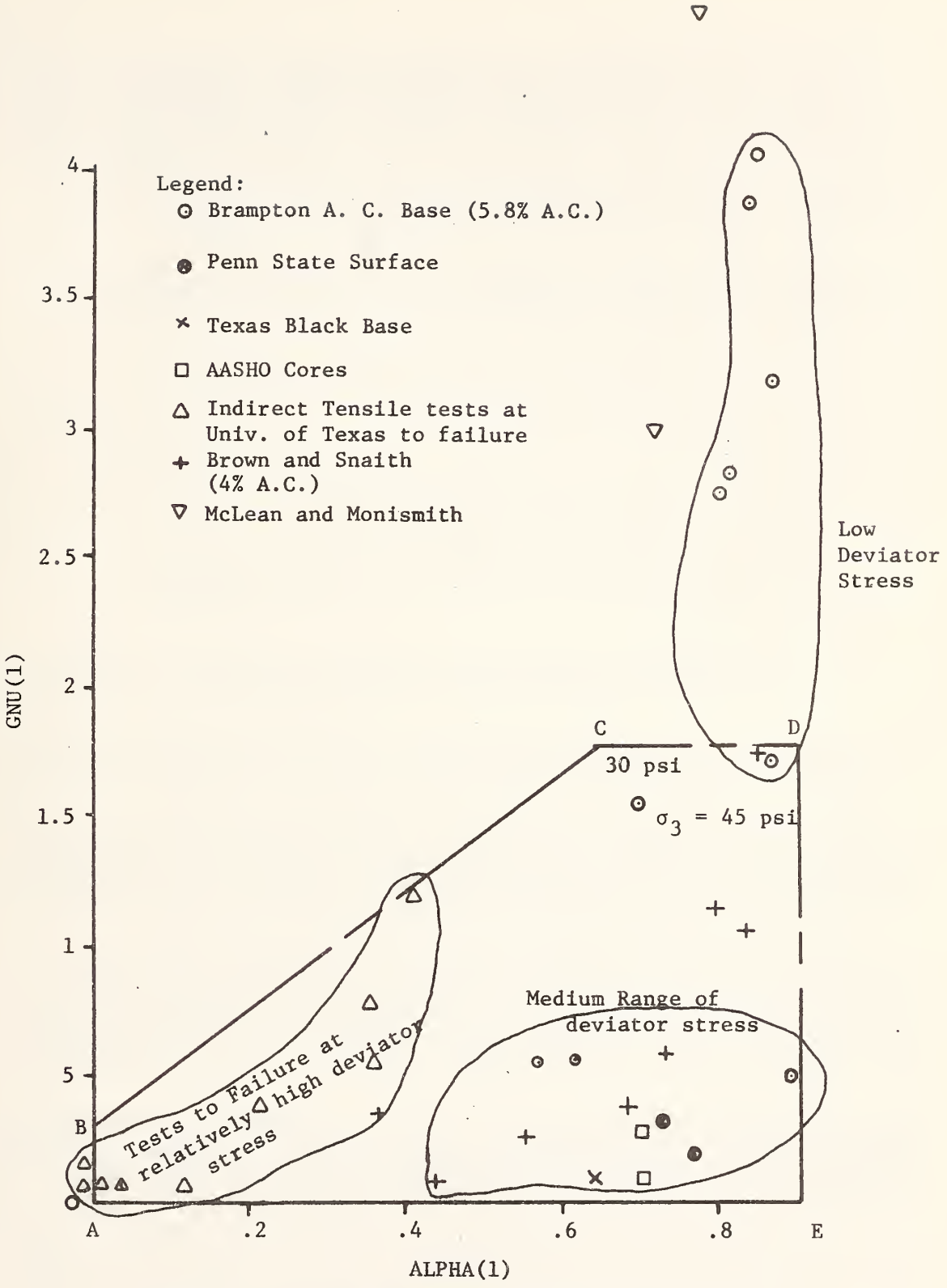


Figure 57. Relation between ALPHA(1) and GNU(1)

possible and quite useful to perform a more detailed multiple regression on ALPHA(1) and GNU(1) utilizing deviator stress, temperature, mix parameters, etc. as independent variables. Equations could be developed to provide estimates of ALPHA(1) and GNU(1) and further study could establish the relative importance of the independent variables. Considerable data exist for such a study, but a test program would be required to broaden the ranges for some of the variables.

Analysis of Causes for Overprediction of Rutting By VESYS IIM, Refinement of ALPHA(1) and GNU(1)

The input parameters that affect rutting include:

1. Thicknesses of Layers 1 and 2
2. Traffic
3. Radius and Amplitude of Wheel Load
4. Creep-compliance Relations for the Three Layers
5. Temperature
6. The Permanent Deformation Coefficients ALPHA(1) and GNU(1) for Layer 1 and ALPHA(3) for the subgrade.

Although a very considerably effort had gone into defining these parameters, rutting as is still overpredicted by VESYS IIM.

One previous source of problems was the unrealistic values of Layer 1 creep-compliance (layers not stiff enough) in the literature, but more realistic values were developed and reported in Appendix B. This had the effect of reducing both rutting and cracking.

While the study reported in the previous section considerably expanded available knowledge of the permanent deformation coefficients ALPHA(1) and GNU(1), application of this knowledge had the effect of increasing rutting as lower values of ALPHA(1) believed to apply for the range of stress conditions under study.

Review of the testing procedures for obtaining ALPHA(1) and GNU(1) and of their use in the computer program brought two things into focus that may account for a portion of the overprediction. These are listed below and subsequently discussed in more detail.

1. Testing for ALPHA(1) and GNU(1) has been conducted only to 10,000 to 20,000 cycles on the assumption that the ratio of permanent to elastic deformation has stabilized. This is not a valid assumption.

2. The relationships for ALPHA(1) and GNU(1) depicted in Figures 49 and 50 depended altogether on indirect tensile test data for deviator stresses higher than 50 psi, except that the Brown and Snaith data developed with triaxial compression testing included deviator stresses to about 130 psi. The low levels of ALPHA(1) at higher deviator stresses developed from the indirect tensile test data yield unrealistically high rut depths, which raises doubts as to the values obtained. The state of stress in the indirect tensile specimen is much different from that in a triaxial compression specimen, which means that the rate of permanent deformations with dynamic loading could also vary considerably. (Subsequent values for ALPHA(1) and GNU(1), using vertical instead of horizontal strains for indirect tensile test specimens at the University of Texas at Austin were fairly consistent with data from compression tests, but were not available in time for use in this work).

Testing and Possible Extrapolations for Asphaltic Concrete ALPHA(1) and GNU(1)

Fortunately, considerable testing for permanent deformations for asphaltic concretes by Morris at The University of Waterloo (Ref 16) has been carried out to 700,000 cycles of loading. This allows comparisons of permanent deformation coefficients when calculated for 10,000 and 30,000 cycle data and for 300,000 and 700,000 cycle data. Data points for a typical test in the Morris series on asphaltic concrete base used in the Brampton Road Test appear in Figure 58 at load cycles of 10,000, 30,000, 100,000, 300,000, and 700,000. The line identified as Number 1 was drawn through the 300,000 and 700,000 cycle data points and produced an ALPHA(1) of 0.69 and a GNU(1) of 1.94. By using the data points for 10,000 and 30,000 cycles, Line Number 2 is obtained and ALPHA(1) and GNU(1) are 0.35 and 0.084, respectively.

To further develop this line of thought, the curve from the measured data is extrapolated back to the first cycle in a manner typical of measured data from other tests. If it is understood that the predicted accumulation of permanent deformations follows the line out with increasing load cycles, it can be seen that either Line 1 or Line 2 will overpredict permanent deformations for the first 10,000 to 100,000 cycles. The selection of this means of curve fit accepts this as a potential inaccuracy on the basis that the number of cycles

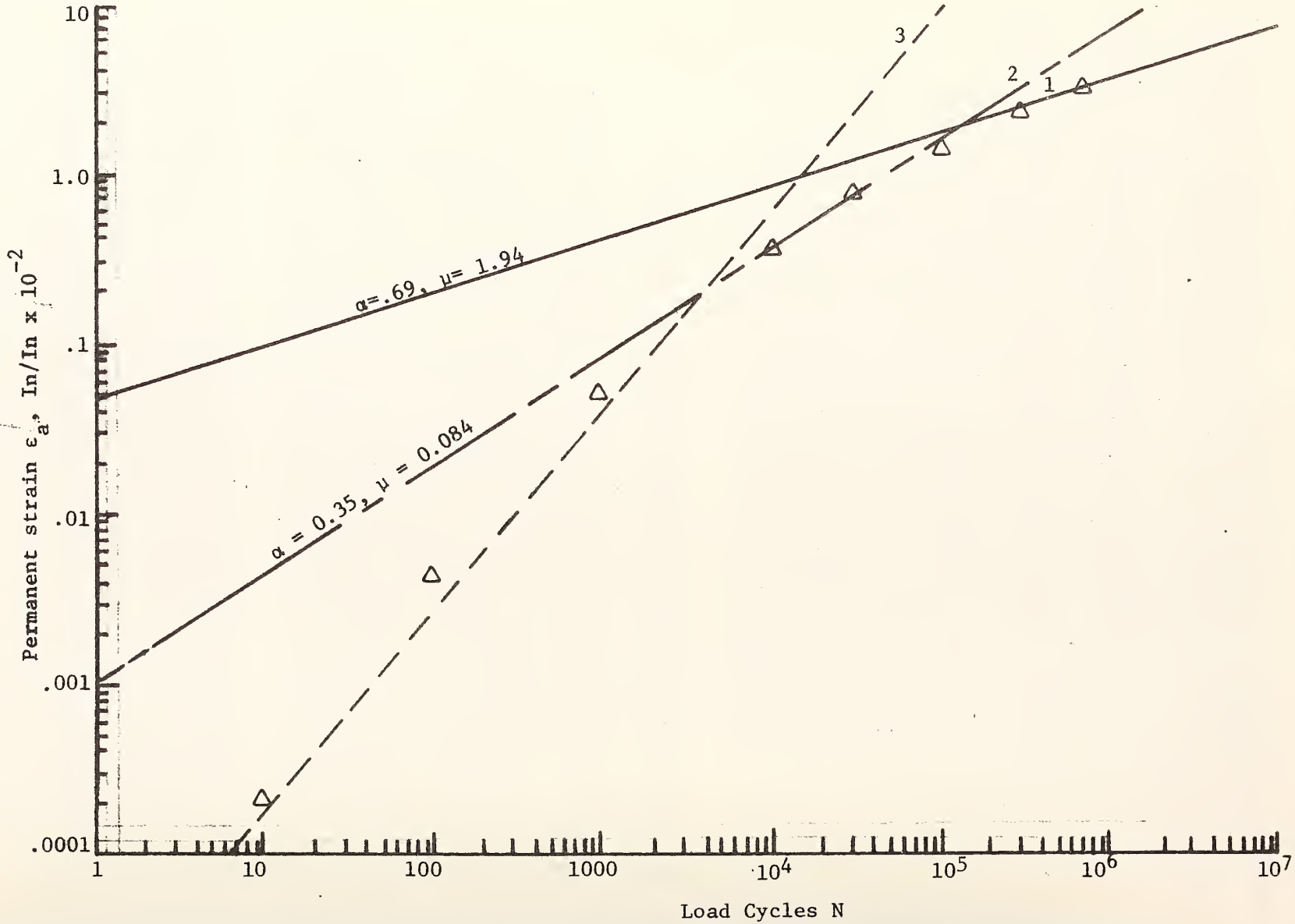


Figure 58. Permanent deformations versus load cycles plotted through different data points for a Brampton Road Test Mix.

involved is minor compared to the total traffic so that the accumulated overpredictions represent an acceptable error in rutting predictions after say one million load cycles. If Line 1 is taken to be the more accurate, it can also be seen that Line 2 begins to increasingly overpredict permanent deformations after about 100,000 cycles.

It may be possible to obtain a better method of fitting this data, but doing so would involve major revisions to VESYS IIM. The best utilization of effort appears to be better adaptation of the curve fitting approach.

It appears rather necessary to obtain test data out into the range of one million or so cycles by some means. An actual test to carry through a million cycles would last 5 or 6 days for typical load and rest periods. This would prove to be a costly test procedure, but not near as costly as that required to obtain the fatigue relationship. An alternate might involve testing to some level such as 30,000 cycles and extrapolating the data out to one million cycles. Fairly simple relations could be developed for the extrapolation from a test program using a range of mixes and testing each to one million cycles under several different stress conditions.

Initial insight into the possible effects of such extrapolation may be obtained by considering the information in Table 15, calculated from ten tests reported in Ref 16 which provides values of ALPHA(1) and GNU(1) when Line 1 and Line 2 are used as plotted in Figure 58. Also included are predictions of permanent strain at one million cycles for Line 1 and Line 2 and comparisons. Note that the overpredictions of permanent deformations varied from -4% to 278%.

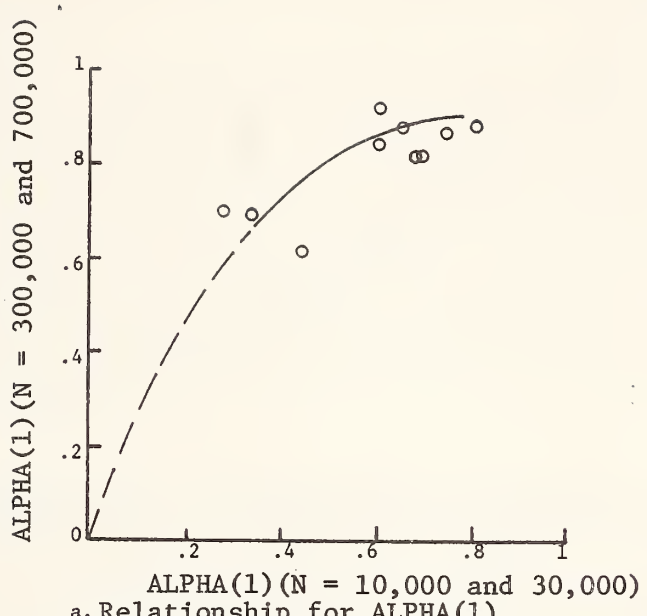
The values of ALPHA(1) and GNU(1) for Line 1 have been plotted versus the same values for Line 2 in Figure 58. The approximate relations plotted are indicative of what might be expected from a more detailed study to develop relations to extrapolate ALPHA(1) and GNU(1) from low load cycle tests to higher load cycle conditions.

The relation in Figure 59a indicates that ALPHA(1) at the high values of load cycles may be twice that for the lower values of load cycles up to a value for the latter of 0.3 (implies high deviator stresses), while the difference decreases as ALPHA(1) increases until they are approximately equal for an ALPHA(1) of 0.85 (implies very low stresses).

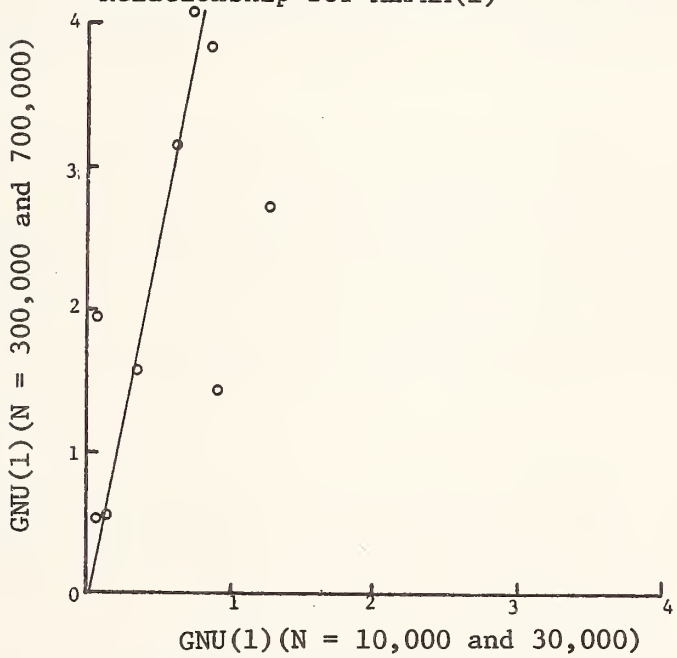
As ALPHA(1) is strongly dependent on deviator stress and deviator stress is strongly dependent on pavement thickness and bending stiffness, it would be possible to obtain a fairly simple relationship for transforming data from short-term tests to higher levels of loading cycles. This could be handled internally by the computer program and the increases could be tailored to the number of accumulative cycles during the time

Table 15 . Comparisons of permanent deformation coefficients and permanent strain ε_a prediction from lines drawn through different data points, Brampton asphaltic concrete base material.

	Calculations from Line 1 through N = 300,000 and 700,000			Calculations from Line 2 through N = 10,000 and 30,000			$\varepsilon_{a2} - \varepsilon_{a1}$	$\frac{\varepsilon_{a2}}{\varepsilon_{a1}}$
Sample No.	ALPHA(1)	GNU(1)	ε_a^1s at $N^1 = 10^6$	ALPHA(1)	GNU(1)	ε_2 at $N = 10^6$		
01	0.69	1.94	.035	0.35	0.084	.070	.035	2.0
07	0.90	0.51	.0066	0.61	0.076	.0032	.0034	2.06
09	0.81	2.74	.065	0.69	1.28	.042	.023	1.55
12	0.81	2.94	.0019	0.70	0.13	.0015	.0004	1.27
14	0.70	1.56	.135	0.29	0.36	.51	.375	3.78
20	0.61	0.56	.120	0.44	0.11	.170	.050	1.42
21	0.87	3.16	.0053	0.66	0.85	.0033	.0020	1.61
30	0.85	4.05	.0048	0.76	2.27	.0042	.0006	1.14
23	0.84	3.83	.0167	0.62	0.85	.031	.014	1.86
33	0.88	1.43	.00114	0.82	0.92	.0011	-.0003	.96

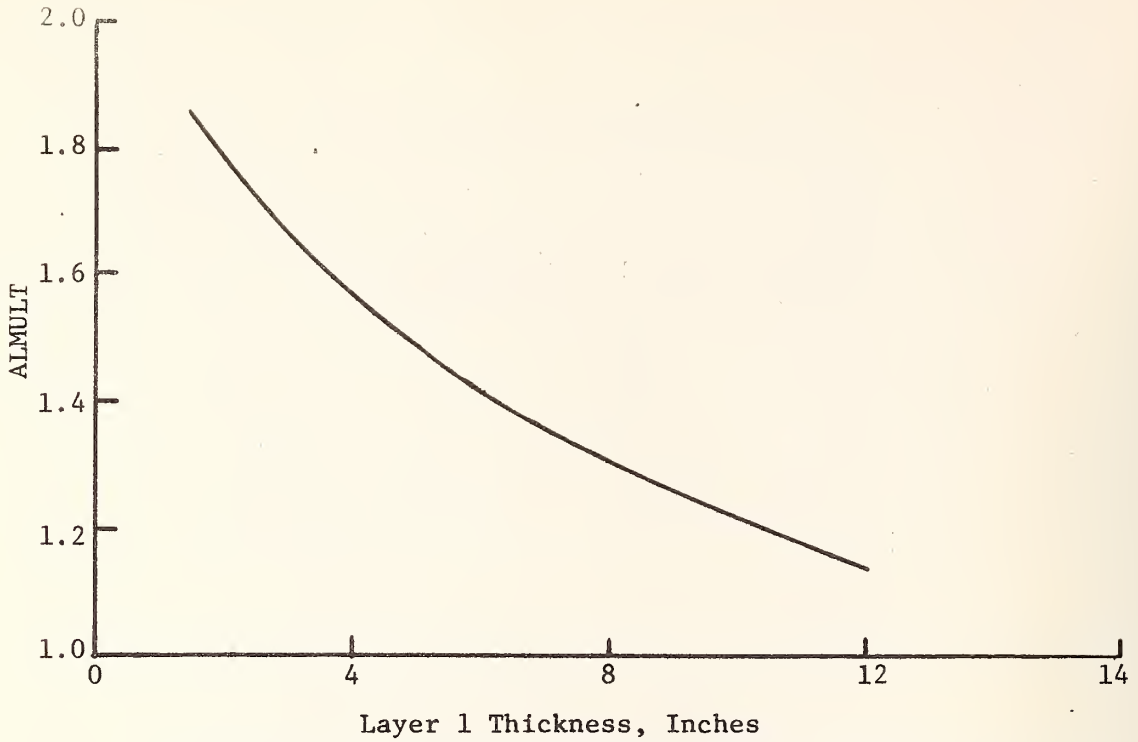


a. Relationship for ALPHA(1)

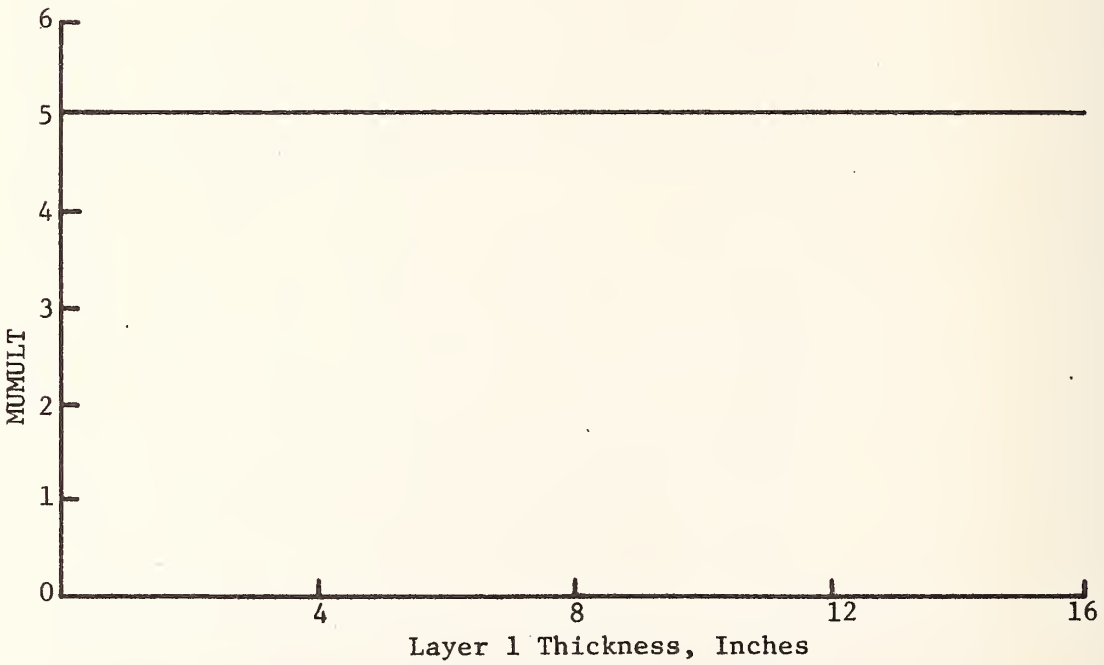


b. Relationship for GNU(1)

Figure 59. Relationship for ALPHA(1) and GNU(1) calculated from data after 10,000 to 30,000 cycles and after 300,000 to 700,000 cycles.



a. Multiplier to increase ALPHA(1) as a function of Layer 1 thickness



b. Multiplier to increase GNU(1) as a function of Layer 1 thickness.

Figure 60. Multipliers to improve rutting predictions.

and temperature cycle under incremental consideration. Figure 60a shows such an example relation developed from Figure 59a and rough approximation of deviator stress with Layer 1 (stiffness omitted). The value of ALMULT would serve as a multiplier to increase ALPHA(1) based on Layer 1 thickness.

Figure 59b indicates that GNU(1) increases greatly as ALPHA(1) is increased for higher numbers of load cycles. This is rational as increasing ALPHA(1) implies less slope for the line, and the intercept on the ordinate may be expected to increase at a higher rate than the slope will decrease GNU(1) = (Intercept x slope ÷ resilient strain). Note that the slope for Line 1 in Figure 58 is approximately ½ that of Line 2, but the intercept is 50 times as great. GNU(1) would then be 25 times as great, which compares very well with the ratio of 23 for Sample 01 in Table 15. GNU(1) would need to be increased according to appropriate relationships developed as described for ALPHA(1) above.

Reconsideration of Values of Asphaltic Concrete ALPHA and MU at Higher Deviator Stresses - The approach in general for the previous section was to plot all data available and to study the trends and ranges of values. As it appears that the indirect tensile data may yield unrealistically low values of ALPHA(1), it will be useful to plot trends when possible for individual test series. This has been done for ALPHA(1) and GNU(1) in Figures 61 and 62 respectively.

Extrapolation of the curve for the Brampton data to follow the trend of the Brown and Snaith data yields the dashed line in Figure 61 (Note that the McLean and Monismith data and the AASHO Core data generally fall at lower values of ALPHA(1) as might be expected because it was based on 10,000 to 20,000 loading cycles as previously discussed). While this is an extrapolation, it is supported by the shape of the Brown and Snaith curve at a much lower asphalt content. Within the 50 to 60 psi average deviator stress of interest for the sensitivity analysis (see hatched zone in Figure 4), ALPHA(1) may vary from 0.5 to 0.9.

Similar treatment for GNU(1) depicted in Figure 62 did not develop any important change in the values for GNU(1). The hatched zone of interest for the sensitivity analysis indicates that GNU(1) may vary from 0.2 to 1.5.

Further Refinement of ALPHA(1) and GNU(1)

Study of ALPHA(1) and GNU(1) to this point has been aimed at learning as much as possible from test results about ALPHA(1) and GNU(1) with the intent of assigning rational and realistic values for input for the sensitivity analysis. This has led to establishment

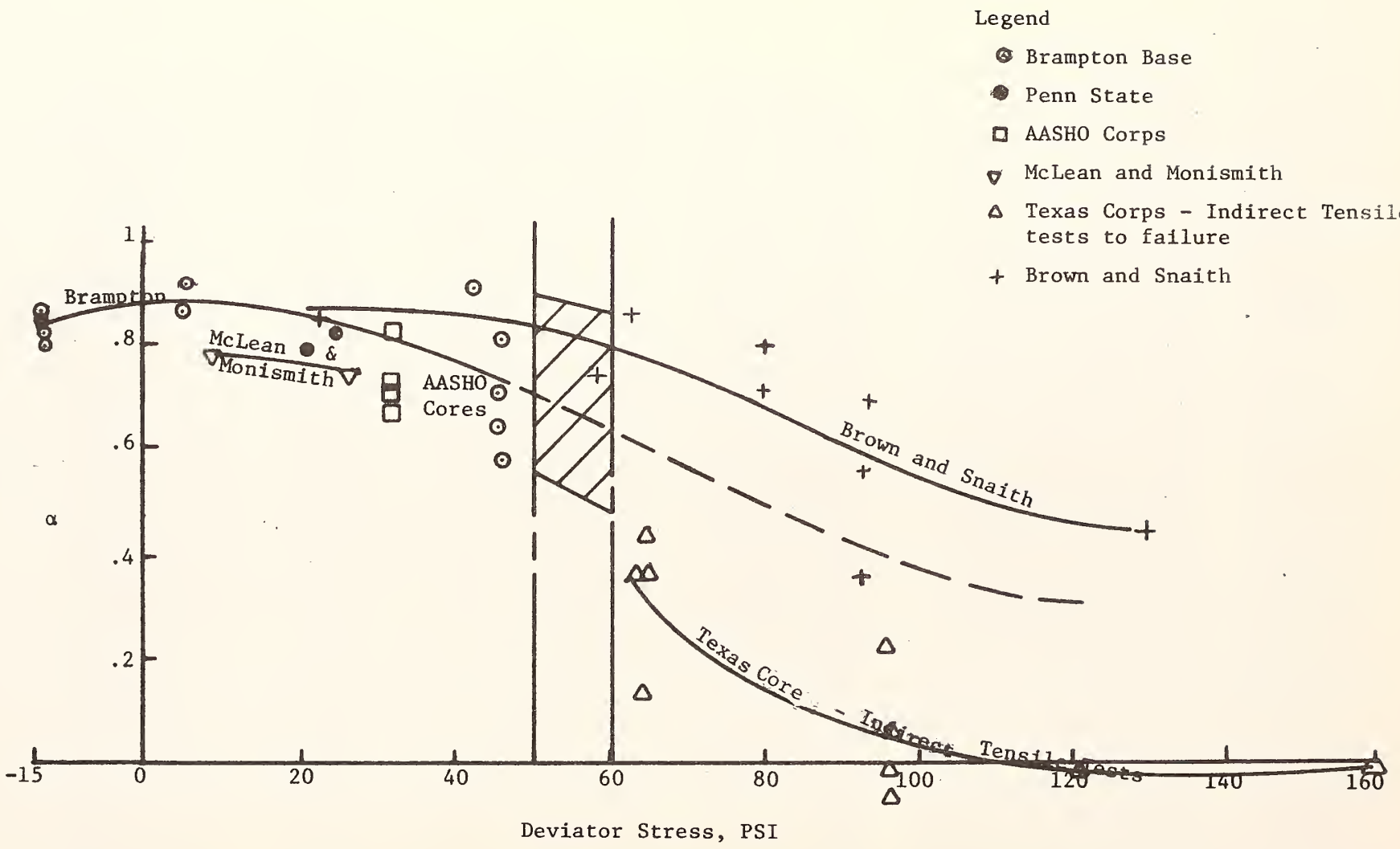


Figure 61. Plot of ALPHA(1) versus deviator stress for asphaltic concrete.

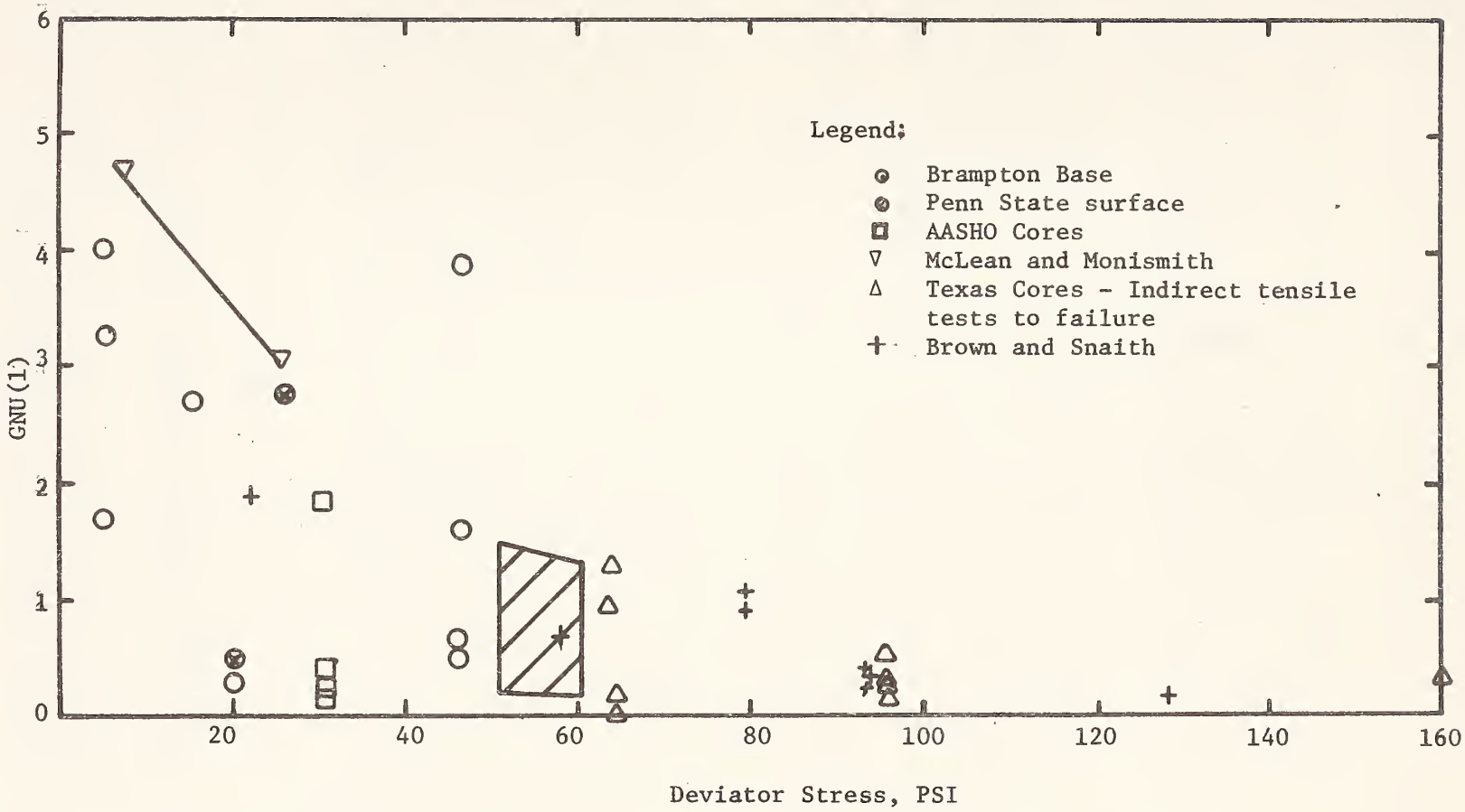


Figure 62. Plot of GNU(1) versus deviator stress for asphaltic concrete.

for the pavement thicknesses and conditions of interest of a range of ALPHA(1) of .5 to .9 and for GNU(1) of 0.2 to 1.5. It is also known that:

1. A low ALPHA(1) or a high GNU(1) increase rutting and vice versa.

2. Although quite variable, a low GNU(1) is usually associated with a low ALPHA(1) and vice versa. This means that for a particular material whose permanent deformation characteristics under repeated loading are defined by ALPHA(1) and GNU(1), the effects of GNU(1) and ALPHA(1) tend to compensate.

Study to date indicates that the VESYS IIM procedure is rational in predicting permanent deformations due to vertical consolidation. This overlooks the really quite important contribution from permanent horizontal displacements, but this is approximately taken into account by using average deviator stress across the Layer 1 thickness. In effect, the tensile horizontal component of stress in the lower portion of the layer is additive to the vertical component, resulting in large deviator stresses and higher rutting.

Having essentially exhausted the possibilities for "bracketing" ALPHA(1) and GNU(1) from test data, it appears logical to now see how VESYS IIM responds to ALPHA(1) and GNU(1) to further "bracket" their values for the sensitivity analysis.

Using input values for each input parameter at the typical or average values established for the preliminary sensitivity analysis except for ALPHA(1) and GNU(1) a small factorial was run varying ALPHA(1) (0.6, 0.75, 0.9) and GNU(1) (0.2, 0.8, 1.5) to gain insight into the variation of responses over this range established as previously described. Values of ALPHA(1) below 0.6 were not included as experience with VESYS IIM indicated fairly excessive rutting below this range and because testing appears to underpredict ALPHA(1) due to limitations on numbers of cycles. The results of this factorial appear in the plots for responses after 5 years in Figures 63 and 64 Other periods of time were available but the trends were approximately the same so these were taken as typical. Damage index (cracking) was not included as it is calculated by a separate model and is unaffected by ALPHA(1) and GNU(1).

Some important results from analysis of these plots are:

1. All responses (except cracking, of course) are highly sensitive to ALPHA(1) and GNU(1) individually or as a coupled pair.

2. When ALPHA(1) is relatively small (say 0.6 to 0.7), GNU(1) has the greater effect. At higher values of ALPHA(1) (say greater than 0.7), ALPHA(1) has the greater effect.

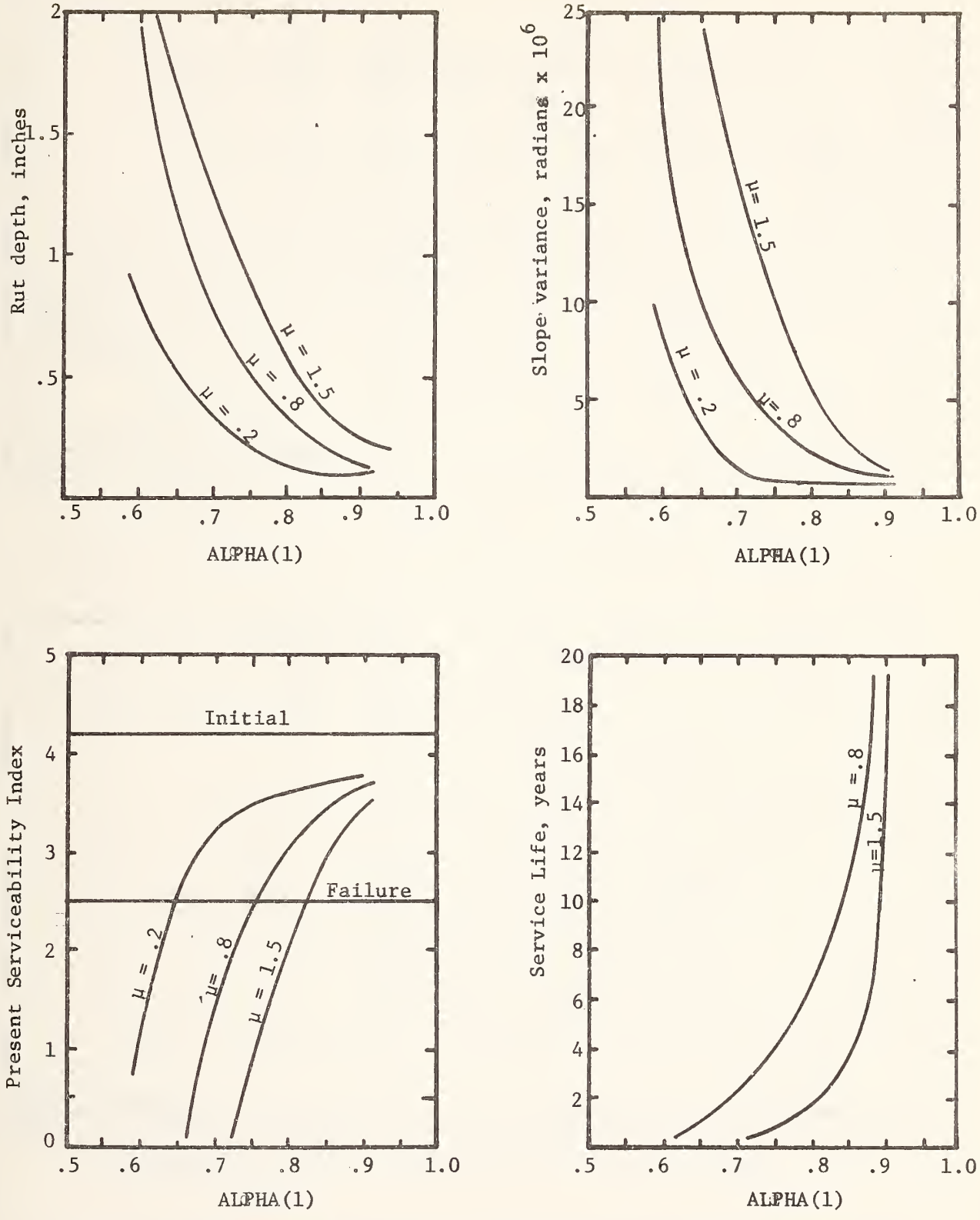


Figure 63. Response variations with ALPHA(1) and GNU(1) after five years viewed with ALPHA(1) as the abscissa.

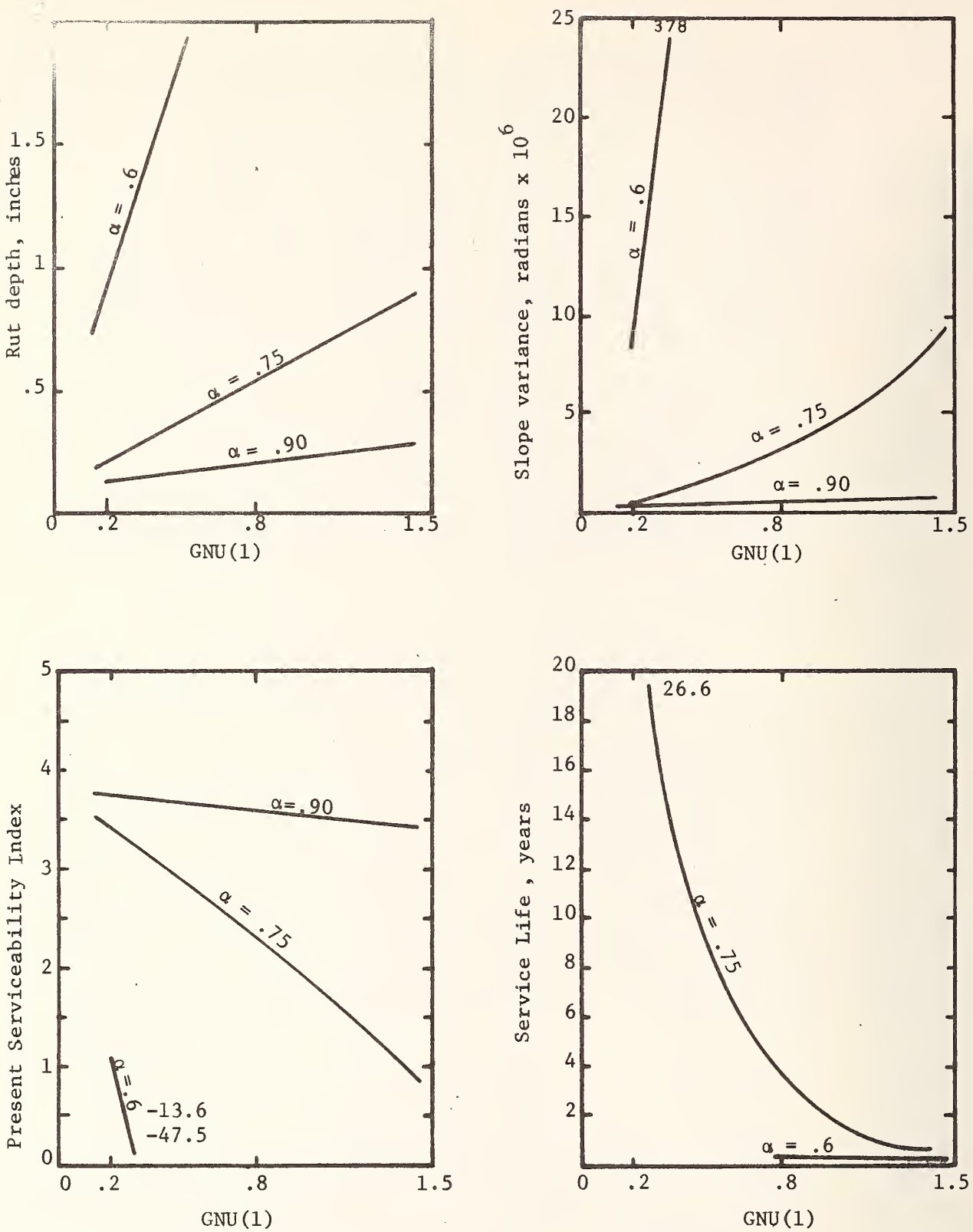


Figure 64. Response varations with ALPHA(1) and GNU(1) after five years viewed with MU on the abscissa .

3. Rut depth varies essentially linearly with GNU(1) and PSI and slope variance vary almost linearly with GNU(1).

4. There is virtually no rutting, slope variance or deterioration (cracking not considered) for ALPHA(1) greater than 0.90.

It is desired to further refine the range of ALPHA(1) and GNU(1) for purposes of the sensitivity analysis to that which will give reasonable calculated responses. To do so, judgement may be applied for several of the reponses and ALPHA(1) and GNU(1) may be obtained at those levels from either Figures 63 or 64. The levels selected as reasonably representative at five years for the pavement system, traffic, environment, etc. described are as follows:

1. Rut depths - 0.2 to 0.4 inches
2. PSI - 2.75 to 3.5
3. Expected life - 5 to 10 years (Independent of Time Period)

Values of GNU(1) and ALPHA(1) were obtained for these levels and plotted in Figure 65. Note that these plots fall in a rather narrow band and that the plots for the poorer and better performances fall in two separate very narrow bands.

It is very important to note that these bands are consistent with the region in Figure 57. within which test data for medium level deviator stresses (type of interest) fell. This implies that the reponses for VESYS IIM for reasonable values of ALPHA(1) and GNU(1) may calculate reasonable responses.

It is equally important to note as previously mentioned that only values of ALPHA(1) and GNU(1) falling near or within the bounds of the plots in Figure 65. may be considered consistent with the asphaltic concrete mixes used in practice. As a counterexample, an ALPHA(1) of 0.65 and a GNU(1) of 1.5 might describe an unstable mix with unrealistically high asphalt content of high penetration asphalt cement. From Figure 63 , rutting would be 2 inches in 5 years, PSI would be a negative 2.5 and service life virtually nonexistent. At the other extreme, an ALPHA(1) of 0.88 and GNU(1) of 0.2 might describe a very stiff mix that would rut less than 0.1 in 5 years and suffer virtually no loss of serviceability. Its service life appears from the plot to be in excess of 50 years.

The mixes represented by combinations of ALPHA(1) and GNU(1) much outside the band are not necessarily impossible, but should not generally be expected when modern mix and pavement thickness design procedures are applied.

Although the rationale above greatly reduces the potential region of ALPHA(1) and GNU(1) to be considered, only a few values of ALPHA(1)

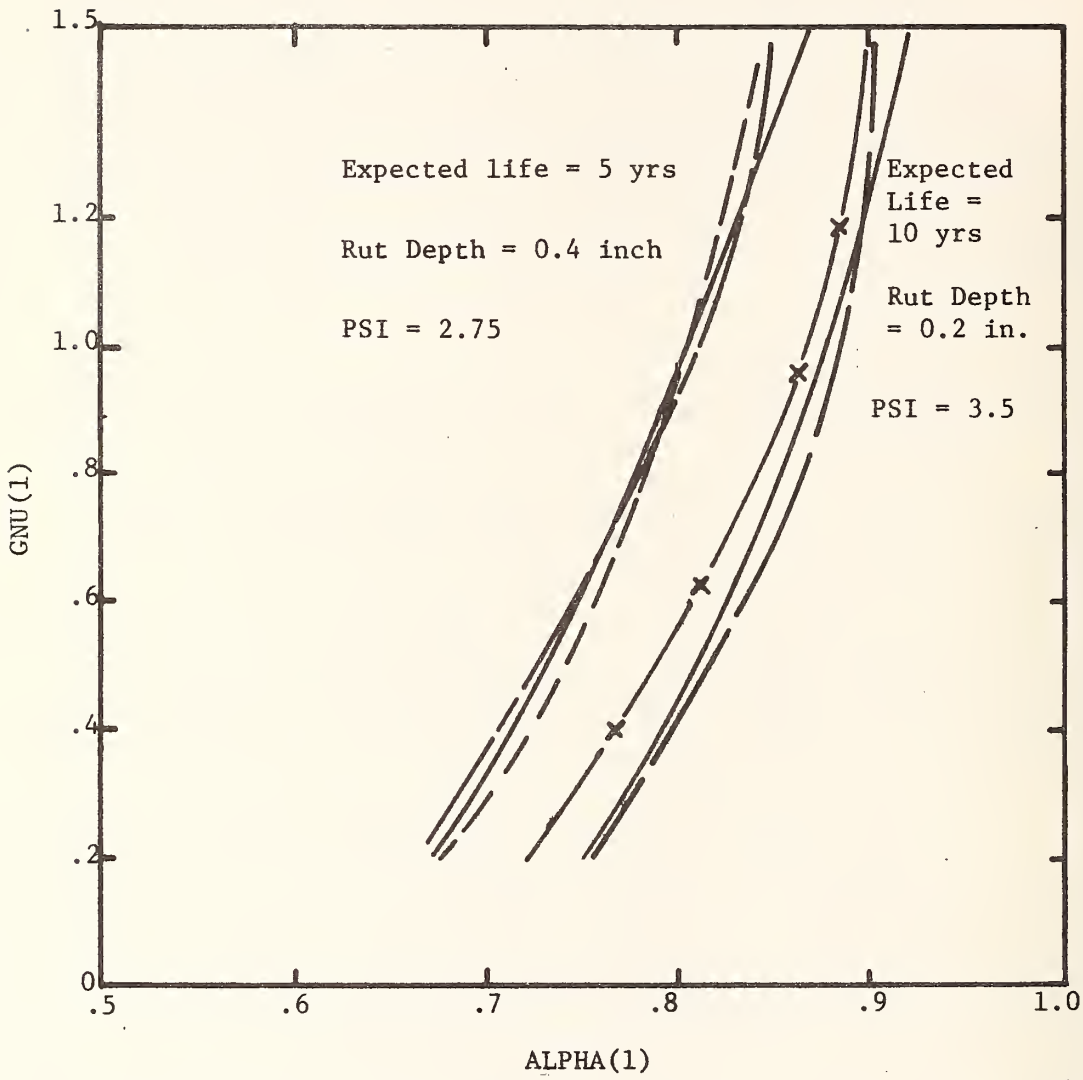


Figure 65. Typical responses plotted as functions of ALPHA(1) and GNU(1)

and two of GNU(1) may be used in the sensitivity analysis, so additional refinement is required. The test data falling in the range of deviator stresses from 40 psi to 70 psi (10 psi either side of range of interest for the sensitivity analysis of 50 to 60 psi) has been plotted in Figure 66 along with the boundaries of the region previously developed from Figure 65. With this limited data, 4 out of 6 fell below a GNU(1) of 0.6.

As most data in the "medium range" of deviator stresses identified in Figure 57 also fell below 0.6, the range of GNU(1) for further consideration for the sensitivity analysis will be reduced to that from 0.2 to 0.6. This reduces the range to that hatched on Figure 66 and implies a reduction in range for ALPHA(1) from 0.68 to 0.82.

The potential range of GNU(1) and ALPHA(1) have now been considerably reduced, but would still allow very poor (3/4 inch of rutting in five years from Figure 63 and very excellent performance (0.1 inch of rutting in five years from Figure 63.

It is necessary at this point to make a difficult decision. Should ALPHA(1) and GNU(1) be considered as separate independent varibles or simply as two curve-fitting parameters that coupled describe the permanent deformation characteristics of a material when subjected to cyclic loading? Reasonable arguments may be made for either approach, but the latter appears attractive because:

1. A permanent deformation characterization is the specific need of the rutting model and ALPHA(1) and GNU(1) were only created as mathematical conveniences for defining a relation between accumulated permanent deformations, number of load cycles and the elastic or resilient strain. This is not to say that they may not have individual physical significance, but that such significance is not yet fully defined and the interpretation of either ALPHA(1) or GNU(1) interacting with some other independent variable is less meaningful than an interaction of permanent deformation potential with some other independent variable such as traffic, temperature, pavement thickness, etc.

2. As a secondary consideration, use of ALPHA(1) and GNU(1) as pairs allows at least two other independent variables for consideration in the analysis.

As a compromise to avoid loss of possible information from "coupling" the values, three values of ALPHA(1) and two values of GNU(1) were used in the serviceability analysis to represent approximately the corners of the hatched region in Figure 66. The four sets of values resulting were:

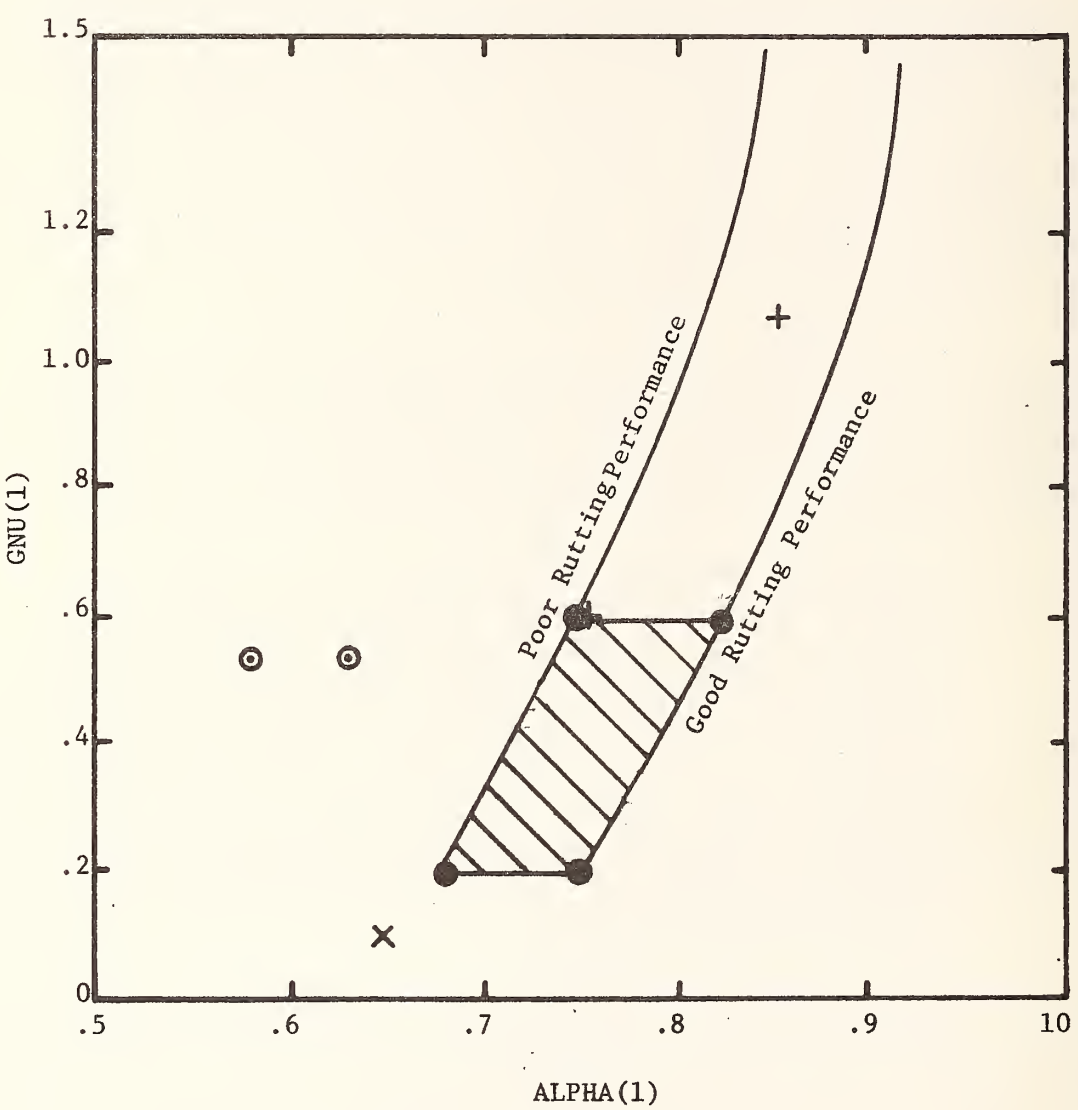


Figure 66. Plot at practical region of ALPHA(1) and GNU(1) couples versus test data

1. ALPHA(1) = 0.68, GNU(1) = 0.20
2. ALPHA(1) = 0.75, GNU(1) = 0.20
3. ALPHA(1) = 0.75, GNU(1) = 0.60
4. ALPHA(1) = 0.82, GNU(1) = 0.60

The values used for predicting performance for the AASHO Road Test sections were based partly on testing and partly on considerations discussed above. At the time of testing, 15,000 to 20,000 load cycles were believed to be adequate. Consequently, the values of ALPHA(1) may be expected to be low and were revised according to the tentative relations between short-term and long-term test results shown in Figure 60. The results of the testing and the values actually used appear below:

	Test Values		Values used	
Section No.	ALPHA(1)	GNU(1)	ALPHA(1)	GNU(1)
266	0.67	0.16	0.87	0.64
272	0.83	1.76	0.89	7.00
308	0.70	0.11	0.88	0.44
470	0.71	0.30	0.88	1.20

Granular Base Material

Figure 44 reflects available permanent deformation data for granular materials that may be reduced to values for ALPHA(2) and GNU(2). As discussed previously, most of the strain response to a cyclic load is elastic and there is little permanent deformation. Accordingly, ALPHA(2) is generally greater than 0.9, but may drop to somewhat smaller values if insufficiently dense or if approaching failure.

GNU(2) generally was less than 0.4, but could range much higher as indicated in Figure 44.

The values used for the preliminary sensitivity analysis were 0.84 to 0.96 for ALPHA(2) and 0.10 to 0.42 for GNU(2). As the calculated responses were found to be relatively insensitive to both ALPHA(2) and GNU(2), development of a mean and standard deviation were not required.

The values of ALPHA(2) and GNU(2) used for predicting performance of AASHO Road Test sections were arrived at by averaging the results of duplicate tests on the base and on the subbase materials and then combining these on a weighted basis dependent on layer thickness. The equation for calculating ALPHA(2) follows:

$$\text{ALPHA(2)} = \frac{\text{Base Thickness(Av.Base ALPHA(2))+Subbase Thickness(Av.Subbase ALPHA(2))}}{\text{Base Thickness + Subbase Thickness}}$$

The value for GNU(2) is calculated in similar manner. As an example for section 266:

$$\text{ALPHA(2)} = \frac{9\left(\frac{.94 + .92}{2}\right) + 16\left(\frac{.97 + .91}{2}\right)}{9 + 16} = 0.94$$

$$\text{GNU(2)} = \frac{9\left(\frac{2.68 + 2.62}{2}\right) + 16\left(\frac{.15 + 1.99}{2}\right)}{9 + 16} = 1.64$$

The resulting values used and basis for their development were:

Section No.	Depth In.	Test Values Base ALPHA(2)	Base GNU(2)	Depth In.	Subbase ALPHA(2)	Subbase GNU(2)	Weighted Values Used ALPHA(2)	Used GNU(2)
266	9	.94	2.68	16	.97	.15	0.94	1.64
		.92	2.62		.91	1.99		
272	9	.94	2.68	8	.97	.15	0.93	1.91
		.92	2.62		.91	1.99		
308	6	.94	2.68	12	.97	.15	0.94	1.60
		.92	2.62		.91	1.99		
470	6	.94	2.68	8	.97	.15	0.94	1.75
		.92	2.62		.91	1.99		

Clay Subgrade Material

Figure 44 reflects available permanent deformation data for clay subgrade materials that may be reduced to ALPHA(3) and GNU(3). ALPHA(3) varied from 0.63 to 1.0 and GNU(3) was generally less than 0.1, but was 3.49 for one AASHO sample.

It was necessary for characterization of the undisturbed subgrade samples from the AASHO Road Test pavements to establish very carefully the stress levels for testing. Dynamic testing supported by another contract had already been conducted for resilient moduli so laboratory curves for variations in resilient modulus with deviator stress were available for Layers 2 and 3 and values for dynamic modulus for Layer 1. It was then possible to use elastic layer theory with subgrade modulus of elasticity varied over a range to derive curves of deviator stress with depth into the subgrade shown in Figure 67. Based on study of Figure 67 and judgement, the deviator stresses at a level 18 inches below the top of the subgrade were selected as representative. These deviator stresses were then plotted against the subgrade moduli of elasticity used in their calculation as shown in Figure 68 for AASHO Road Test section 470. The laboratory curve for resilient modulus was

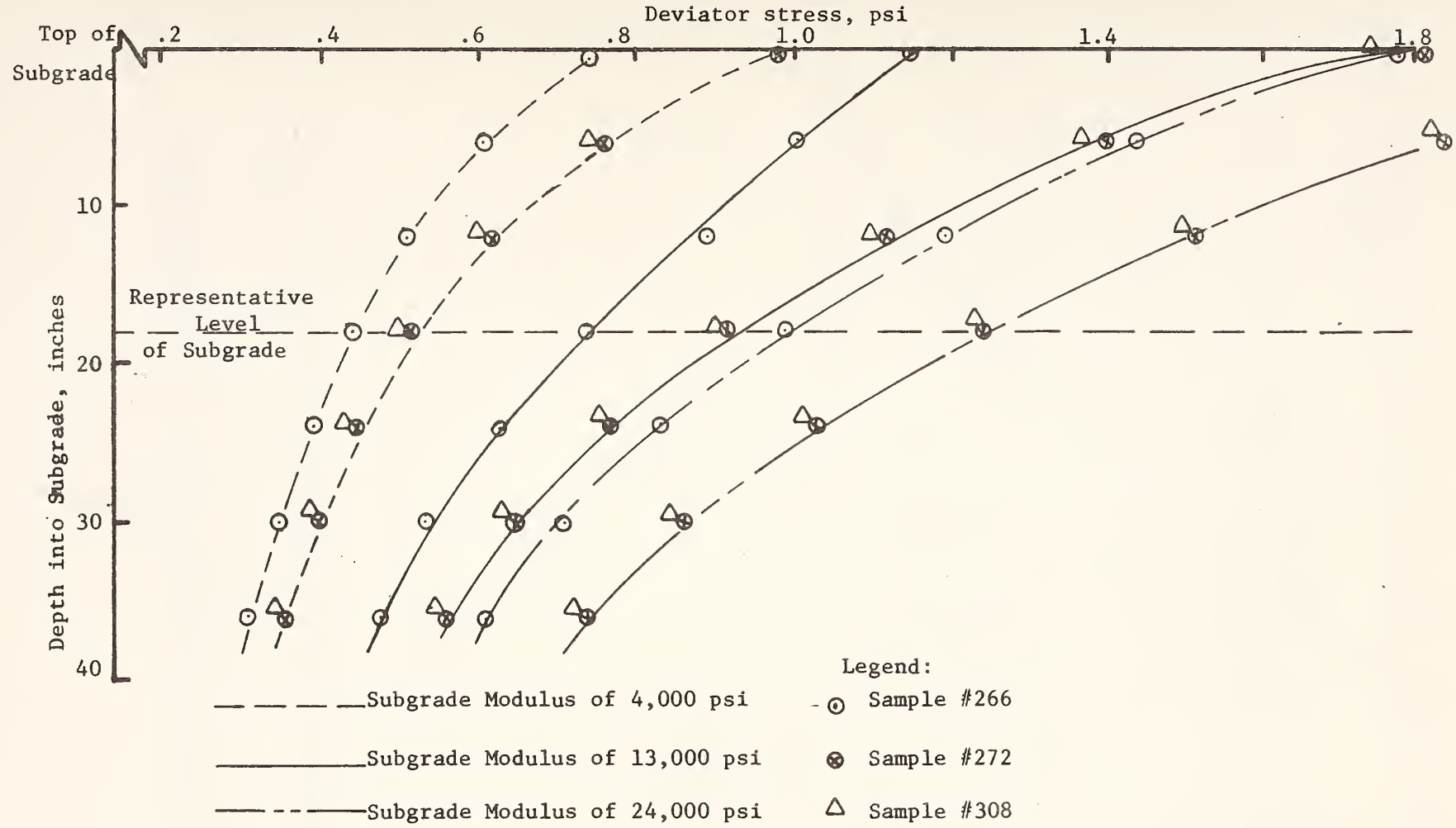


Figure 67. Predicted stress variation with depth for three AASHO Test Road sections and three subgrade moduli of elasticity.

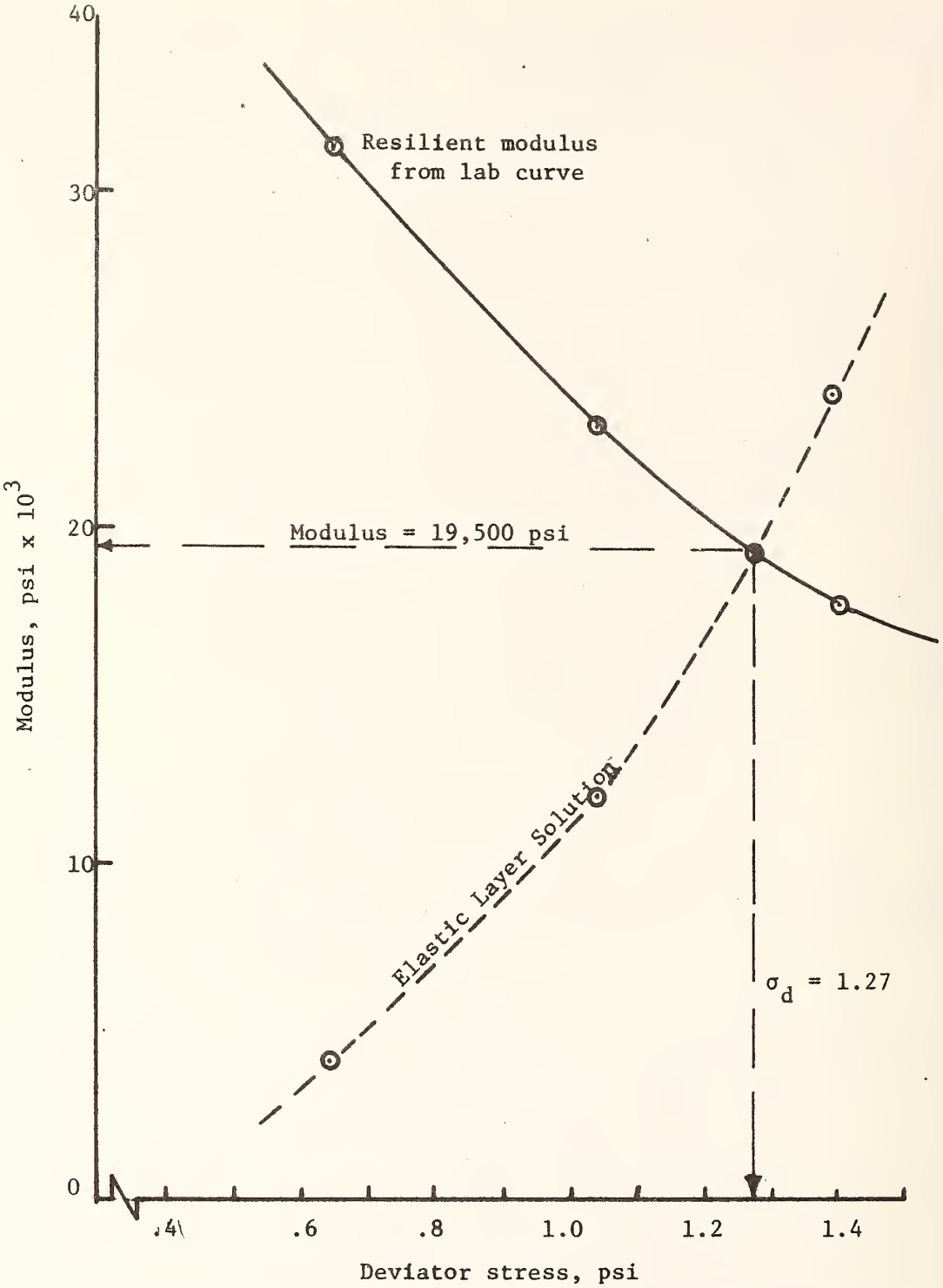


Figure 68. . Modulus of elasticity or resilient modulus versus deviator stress, AASHO Road Test Section 470 subgrade.

also plotted and the intersection of these plots taken to represent the consistent deviator stress for testing. Lateral stresses used were the sum of horizontal stresses calculated due to wheel loads and overburden pressure.

The deviator stresses for these thick pavement structures arrived at by this method ranged from 1.05 to 1.27 psi. While the magnitudes would be higher for thinner pavements, it is believed that most testing has been conducted at higher deviator stresses and that this accounts at least partially for the relatively low values of ALPHA(3) obtained from most other testing compared to 5 out of 6 values .95 or higher obtained on the AASHO soils. (Also, the AASHO soils have already been subjected to a million or more wheel loads).

A range of 0.64 to 1.0 for ALPHA(3) and 0.01 to 0.15 for GNU(3) were used in the preliminary sensitivity analysis. As the responses were found to be insensitive to GNU(3), further effort was not required to develop mean and standard deviation for it. A mean of .815, low of 0.69 and high of 0.94 were used in the main sensitivity analysis for ALPHA(3).

The input values obtained from tests used for predicting performance of AASHO pavements were:

Section No.	ALPHA(3)	GNU(3)
266	0.97	0.10
272	1.00	0.00
308	0.97	0.085
470	0.79	0.120

APPENDIX D

DEVELOPMENT OF REALISTIC CHARACTERIZATIONS FOR FATIGUE LIFE POTENTIAL OF ASPHALTIC CONCRETE

It was clear from the beginning that either the fatigue model in VESYS IIM was not satisfactory or that the fatigue characterizations used were not realistic as all preliminary solutions predicted total cracking within less than one year.

Review of the formulation revealed one error in sign for the stochastic equation for expected damage index $E[D_i]$, which is described in Chapter IV and was corrected. A more serious limitation noted late in the research project is described in Chapter VII, but not yet corrected. This is the poor approximation of the function 1/Ni by the Taylor series expansion used in the stochastic Equation (4) described in Chapter II. The divergence that occurs between the function and its approximation as the coefficients of variation for the fatigue coefficients K_1 and K_2 increase is one of the principal causes for the early predictions of fatigue failure. It should be noted that this error becomes small as the coefficient of variation for K_2 becomes small.

However, it is also clear that the prepondence of data from laboratory testing is not realistic for pavements in service and better fatigue relationships have been developed.

Statistical Characterization of Fatigue Life Potential Using Laboratory Fatigue Data

It became clear as the project progressed that the laboratory curves for fatigue available in the literature grossly underestimated service. (This will be discussed later in some detail). However, considerable information on these test results had been accumulated and is reported.

The two variables to be characterized are STRNCOEF and STRNEXP for the fatigue relationship:

$$N_f = \text{STRNCOEF} \left(\frac{1}{\varepsilon}\right)^{\text{STRNEXP}}$$

used by VESYS IIM to predict pavement fatigue life. The calculation of these two inputs from real data was made using the equations

$$\text{STRNEXP} = {}^{1}/_{s}$$

$$\text{STRNCOEF} = N_i E_i^{\text{STRNEXP}}$$

Where the variables are depicted graphically in Figure 6 in Chapter 3 and/or described below:

N = Number of cycles of loading at which cracking failure occurred for some initial strain ε,

N_i = Initial value of N on plot or measured data,

ε_i = Initial strain causing cracking failure at cycle N_i,

N_f = A predicted value of N to produce failure at a particular initial strain ε_f.

The means and standard deviations reported herein are "between project" relationships. In other words, they represent the differences between materials from separate jobs and not from within any one particular job.

There were four main sources of data (References 18,[1] 19,[2] 20[3] and 21[4]) The data represent different testing temperatures, many different aggregate types, many asphalt types and even different means of testing for the strain - fatigue life relationship. The results of the data analysis are:

	MEAN	STANDARD DEVIATION
STRNCOEF	1.08×10^{-5}	3.41×10^{-5}
STRNEXP	3.294	.850

[1] Kennedy, T. W., and Domingo Navarro, "Fatigue and Repeated-Load Elastic Characteristics of Inservice Asphalt Treated Materials", Preliminary Review, Center for Highway Research, U.T. Austin, January 1975.

[2] Pell, P.S., "Characterization of Fatigue Behavior", Symposium on Structural Design of Asphalt Concrete Pavements to Prevent Fatigue Failures.

[3] Monismith, C. L., J. A. Epps, D. A. Kasianchuk, and D. B. McLean, "Asphalt Mixture Behavior in Repeated Flexure, Soil Mechanics and Bituminous Materials Research Laboratory, Report No. TE 70-5, Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California at Berkeley, 1970.

[4] Pell, P.S., and K. E. Cooper "The Effect of Testing and Mix Variables on the Fatigue Performance of Bituminous Materials, Paper for AAPT, Phoenix Arizona, February 1975.

These data are based on 21 separate strain-fatigue life relationships representing many different aggregate and asphalt types but all at approximately 70^{o}. Some curves were at 68^{o} and some at 75^{o}. Table 16 gives a summary of the data used to calculate the mean and standard deviation. As seen from the data above for STRNCOEF, the standard deviation is larger than the mean, thus verifying the large variability in STRNCOEF due to its exponential relation to and strong correlation with STRNEXP.

The STRNCOEF values ranged from a high value of 1.60×10^{-4} to a low value of 2.2×10^{-19}. The STRNEXP values ranged from a high of 6.1 to a low of 2.35. The wide variation in STRNCOEF explains the large standard deviation. The coefficient of variation for STRNCOEF was 315% while the coefficient of variation for STRNEXP was only 25.8%.

The relationship of STRNCOEF and STRNEXP for the various mixes was added to a plot (Fig.69) extracted from Ref. 21 to give an indication as to whether or not the data used were representative. Figure 69 is a plot of the relationship between STRNEXP and Log STRNCOEF and contains not only 70^{o} data but some 50^{o} test data from Pell and Cooper (Ref 21), Kennedy and Navarro (Ref 18), and Sharma and Kenis (Ref 6). As can be seen, it does not fit the curve as well as Pell's data originally plotted, but does cluster around the same relationship.

Relationship Between Fatigue Coefficients and Temperature for Asphaltic Concrete

The fatigue coefficients STRNCOEF and STRNEXP (same as $K_1(T)$ and $K_2(T)$) are used in VESYS IIM as functions of temperatures in the following equation:

$$N_i = K_{1_i}(T)\left(\frac{1}{\epsilon_i}\right)^{K_{2_i}(T)}$$

such that both K_{1_i} and K_{2_i} appear in an array of time increments i and vary with the array temperature T_i corresponding to the time increments i. Each time increment i usually represents a month of the year and T_i is the average temperature for that month.

VESYS IIM had a tentative procedure for automatically varying K_{1_i} and K_{2_i} with temperature, but it had a limited basis and did not perform rationally throughout the usual range of temperature.

A study was initiated to glean from the literature values of $K_1(T)$ and $K_2(T)$ for tests of the same material at varying temperatures, these data were reviewed and screened and tentative relationships developed for varying $K_1(T)$ and $K_2(T)$ with temperature.

Table 16. Summary of laboratory fatigue data for tests at approximately 70°F

Source	Asphalt Contest and Penetration	Temp.	Type Aggregate	STRNCOEF	STRNEXP	ε_i (N=1000)	Dynamic Modulus
Pell	7.97%,45 pen.	70°	30% crushed rock, 53% sand, 8.9% limestone	8.8×10^{-15}	5.1	4.527×10^{-4}	
	6.5%, 70 pen.	70°	42% CR 46.8% sand 4.7% CR	2.2×10^{-19}	6.1	2.816×10^{-4}	
Monismith	7.9%, 40-50	68°	Granite	6.11×10^{-6}	3.38	3.714×10^{-3}	570,000
British 594	6.0%, 85-100	68°	Course Granite	3.20×10^{-5}	2.49	9.773×10^{-4}	228,000
Cal	6.0%, 85-100	68°	Fine Granite	8.91×10^{-7}	2.95	8.554×10^{-4}	273,000
Cal	6.0%, 85-100	68°	Medium Granite	2.87×10^{-6}	2.83	9.585×10^{-4}	288,000
Cal	6.0%, 60-70	68°	Medium Granite	1.12×10^{-7}	3.26	8.864×10^{-4}	318,000
Cal	6.0%, 40-50	68°	Medium Granite	1.03×10^{-10}	4.01	5.772×10^{-4}	656,000
Cal	6.2%, 60-70	68°	Basalt Medium	1.34×10^{-7}	3.22	8.588×10^{-4}	298,000
Gonzales Field	5.9%, 85-100	68°	Shale	2.12×10^{-8}	3.60	1.084×10^{-3}	154,000
Morro Bay Field	5.9%, 85-100	68°	Sandstone Chert	1.60×10^{-4}	7.35	1.783×10^{-3}	328,000

Table 16. Summary of laboratory fatigue data for tests at approximately 70°F (cont.)

Source	Asphalt Contest and Penetration	Temp.	Type Aggregate	STRNCOEF	STRNEXP	ε_i (N=1000)	Dynamic Modulus
Monismith Morro Bay Lab	5.9%, 85-100	68°	Sandstone Chest	7.51×10^{-6}	2.79	1.225×10^{-3}	459,000
Morro Bat Lab	5.9%, 85-100	68°	Sandstone Chest	2.78×10^{-7}	3.38	1.489×10^{-3}	321,000
San Diego Road	85-100	68°	-	1.5×10^{-6}	2.87	8.42×10^{-4}	273,000
Folsom Field	4.9%, 85-100	68°	Igneous Quartzitic	2.21×10^{-6}	2.86	9.409×10^{-4}	362,000
Folsom Field	4.9%, 85-100	68°	Igneous Quartzitic	8.52×10^{-6}	2.70	1.026×10^{-3}	503,000
Folsom Field	4.9%, 85-100	68°	Igneous Quartzit	3.58×10^{-6}	2.80	9.628×10^{-4}	430,000
Folsom Lab	4.9%, 85-100	68°	Igneous Quartzitic	2.81×10^{-8}	3.47	9.105×10^{-4}	151,000
Folsom Lab	4.9%, 85-100	68°	Igneous Quartzitic	9.67×10^{-7}	2.97	9.221×10^{-4}	226,000
Kennedy	5, 6 and 7% 48 specimens	75°	Limestone	2.20×10^{-8}	3.17	4.345×10^{-4}	
Weighted averages	5, 6 and 7% 51 specimens	75°	Gravel	7.361×10^{-9}	2.88	1.363×10^{-4}	

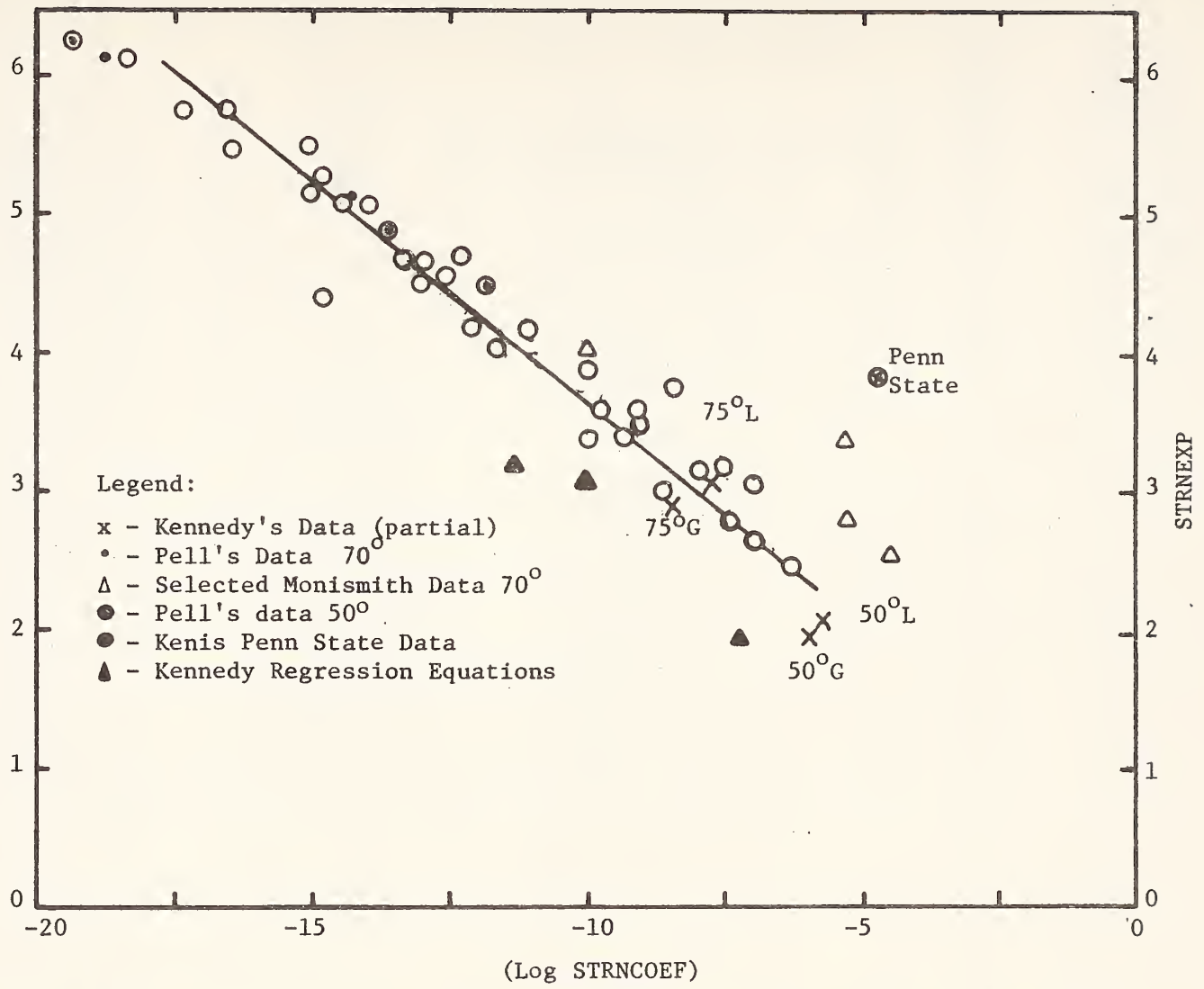


Figure 69. Relationship between K_2 and log K_1 for various mixes (after Pell and Cooper, Ref 21).

There have been many fatigue tests on a variety of asphaltic concrete mixes using several test procedures including the usual uniaxial beam tests, biaxial torsional tests by Pell, biaxial indirect tension tests by Kennedy and others. From this array of data, there are a few cases where sufficient tests have been run with test temperature as the sole variable to begin to understand how K_1 and K_2 vary with temperature.

The system originally used in VESYS IIM assumed that K_1 and K_2 are derived from tests at 70°F. For a particular temperature, these values are then calculated as follows:

$$K_1(T) = K_1(70) \cdot 10^{[.08896\ (T-70) - .0023917(T-70)^2]}$$

$$K_2(T) = K_2(70) - .01349(T-70) + .0004624(T-70)^2$$

These relations were based on one set of data and appear to underestimate fatigue life throughout the temperature range.

Typical log-log plot indicating repetitions to failure for different temperatures as a function of an "initial" strain level (usually measured after 100 to 200 cycles of loading) are shown in Figure 70. The exponent K_2 is the inverse of the absolute value of the slope of the line on the log-log plot. Once K_2 and some point on the line (ε_i, N_i) are known, the following equation may be used to obtain K_1:

$$K_1 = N_i\ \varepsilon_i^{K_2}$$

The data derived from these sources appear in Table 17. As the fatigue curves for the different materials and test procedures vary greatly, it was necessary to normalize the data in relation to its own 70°F curve to allow comparisons. Values of $K_1(T)$ and $K_2(T)$ were divided by K_1 (70°F) and K_2(70°F), respectively to arrived at comparable normalized values. These values also appear in Table 17. Where tests were not run at 70°F, values were taken from the plots in Figures 71 and 72.

The normalized values of K_1 for the data reported in Table 17 are plotted in Figure 73a. If the slope of the fatigue curve may temporarily be assumed not to vary with temperature (variations are usually minor), fatigue life at a specific level of initial strain is then directly proportional to $K_1(T)$, which is simply K_1(70°F) multiplied by the normalized value shown. It can be seen then from Figure 73a that the fatigue life varies across at least seven orders of magnitude as the temperature increases from 0°F to 100°F. Pell's and Taylor's plotted relation appears to imply many more orders of magnitude. The Penn State relation indicates very little fatigue life at low temperature and limited increase in fatigue life at higher temperature.

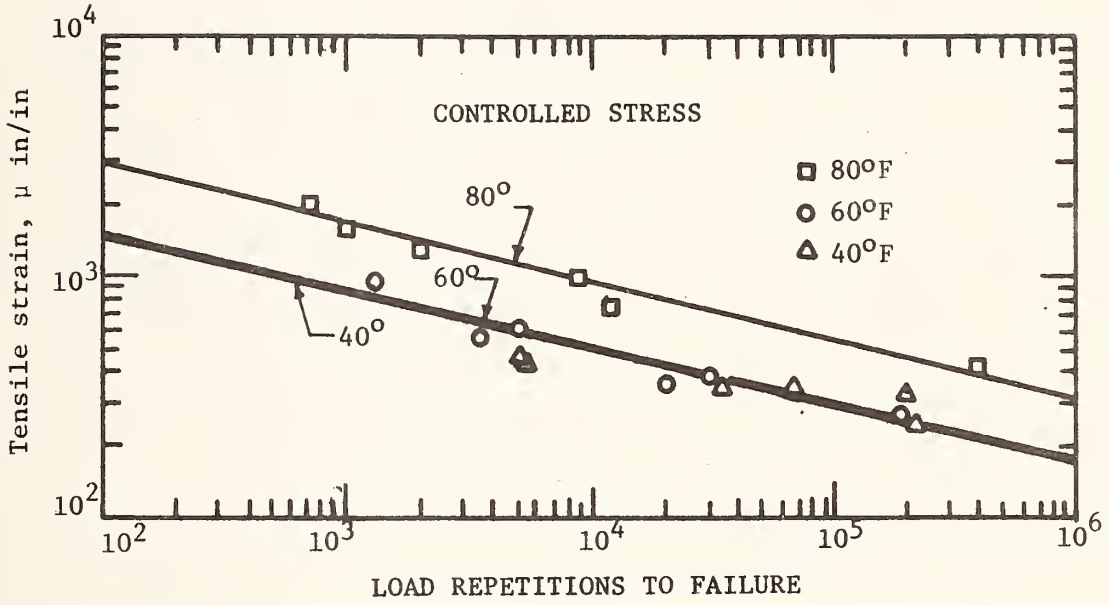


Figure 70. Results of flexural fatigue tests - asphalt concrete surface (from Ref 33).

Table 17. Test data for fatigue coefficients vs. temperature

Source	Type Material	(°F)	K_1	K_2	$\frac{K_1(T)}{K_1(70)}$	$\frac{K_2(T)}{K_2(70)}$
Kingham and Kallas	A. C. Surface	40	$5.45(10^{-11})$	4.32	.19	1.03
		60	$6.31(10^{-11})$	4.32	.22	1.02
		80	$3.12(10^{-9})$	4.14	11.1	.98
	Asphalt treated Base	40	$2.25(10^{-18})$	6.44	.05	1.00
		60	$8.84(10^{-18})$	6.44	.18	1.00
	Sand Asphalt Base	40	$8.83(10^{-10})$	3.83	.05	1.01
		60	$7.49(10^{-9})$	3.81	.41	1.00
Pell and Taylor	Mix G. Base Course 6% Asphalt Gap graded	32	$4.55(10^{-16})$	5.29	.00018	1.20
		50	$4.50(10^{-14})$	4.83	.018	1.10
		68	$3.90(10^{-13})$	4.52	.16	1.02
		86	$2.63(10^{-9})$	3.80	1047	0.86
Kirk	A. C. Surface, Crushed granite and limestone filler	32	$1.87(10^{-18})$	6.43	.013	1.04
		50-59 (same curve)	$2.58(10^{-17})$	6.24	.181	1.01
		77	$6.72(10^{-16})$	6.16	4.70	1.00
Kingham Relation based on AASHO Road Test Data	Developed from AASHO data	5	$2.60(10^{-17})$	5.00	.00009	1.0
		25	$1.48(10^{-16})$	5.00	.0005	1.0
		45	$2.53(10^{-15})$	5.00	.009	1.0
		65	$9.50(10^{-14})$	5.00	.338	1.0
		70	$2.81(10^{-13})$	5.00	1.0	1.0
		85	$4.63(10^{-12})$	5.00	16.5	1.0
		105	$4.03(10^{-10})$	5.00	1434	1.0
Penn State Data	A.C.Surface	55	6.28×10^{-9}	3.92	0.0135	1.09
		70	4.66×10^{-7}	3.61	1.0	1.0
		85	2.90×10^{-6}	3.51	6.22	0.97

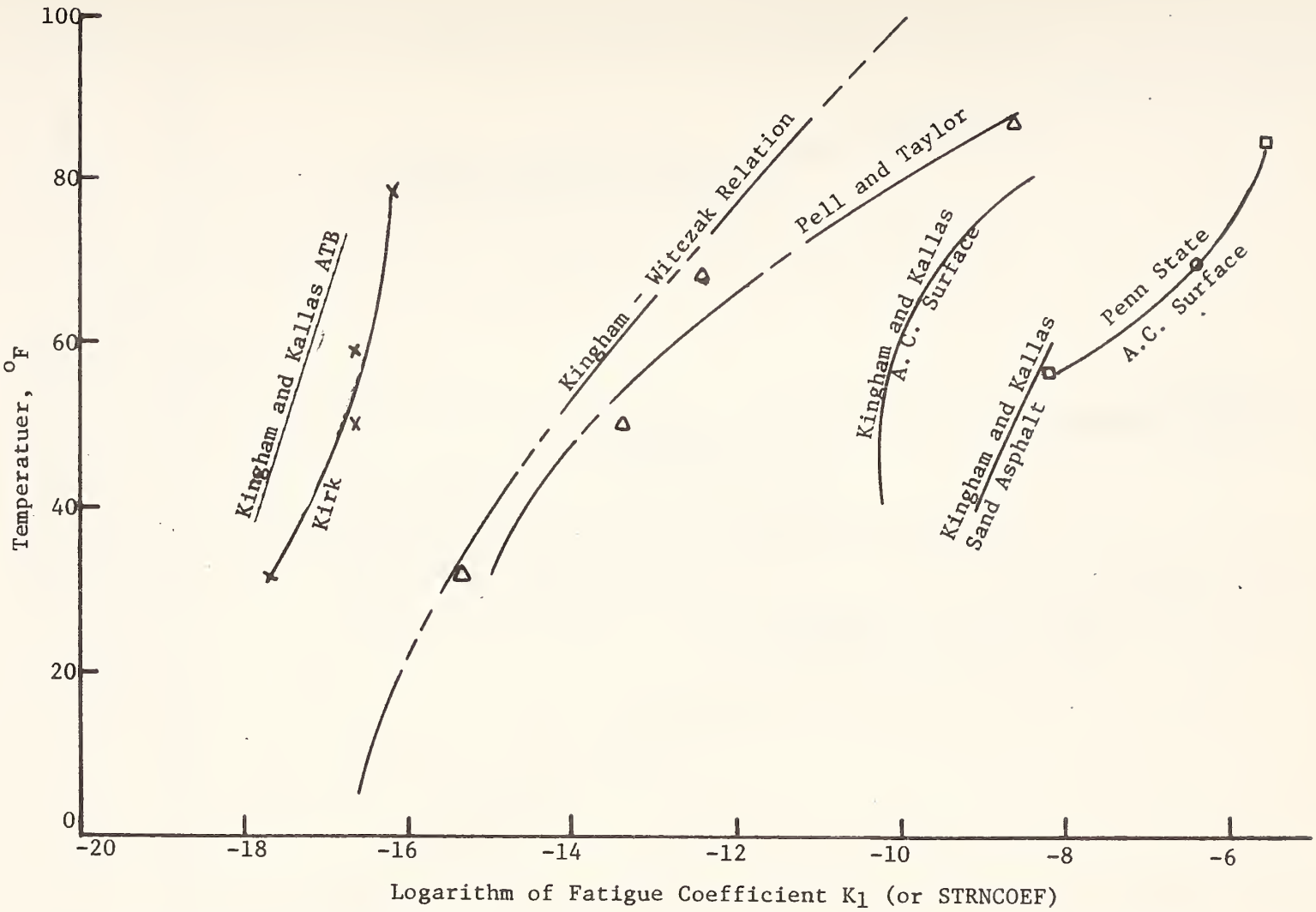


Figure 71. Fatigue Coefficient K_1 as a function of temperature for different mixes.

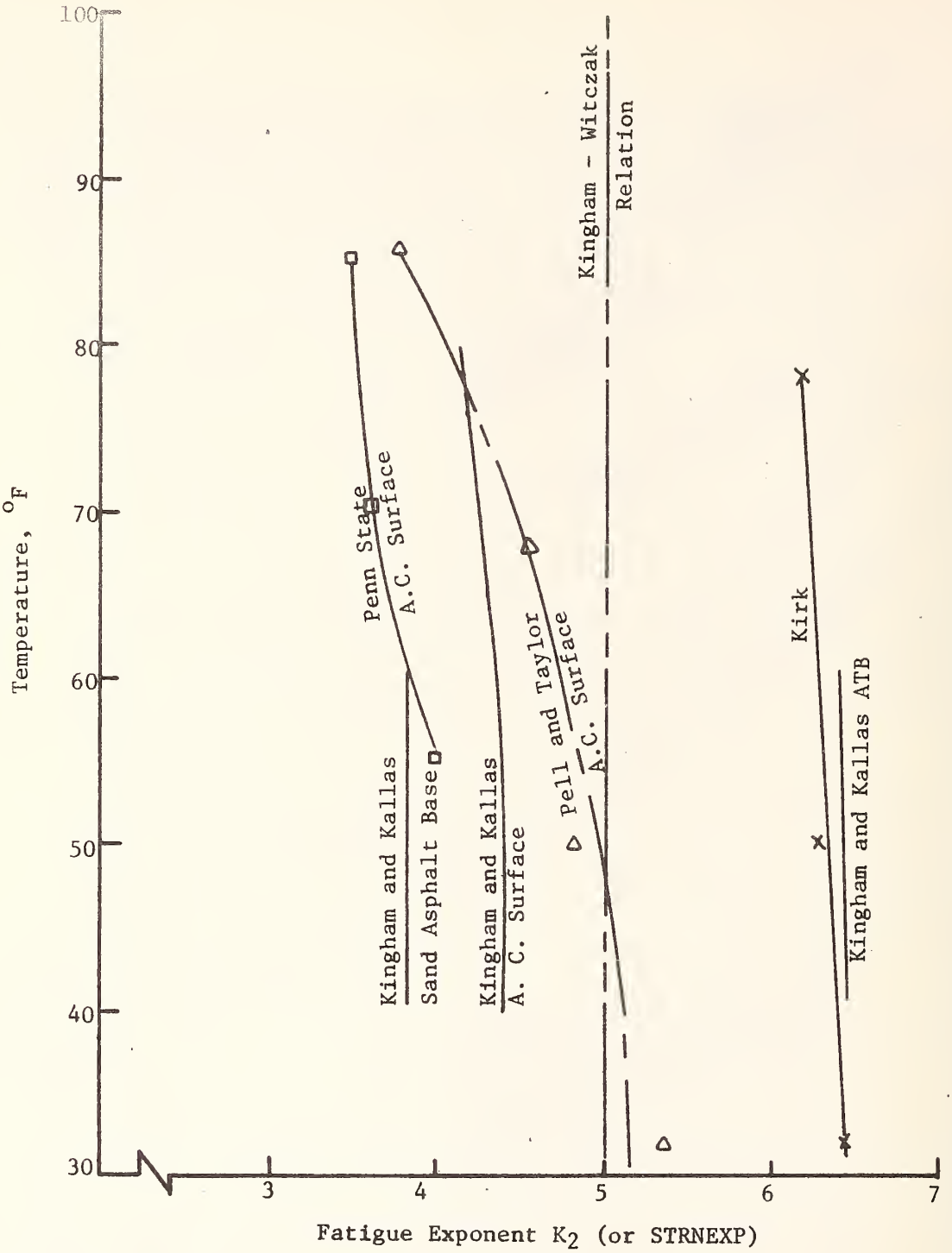
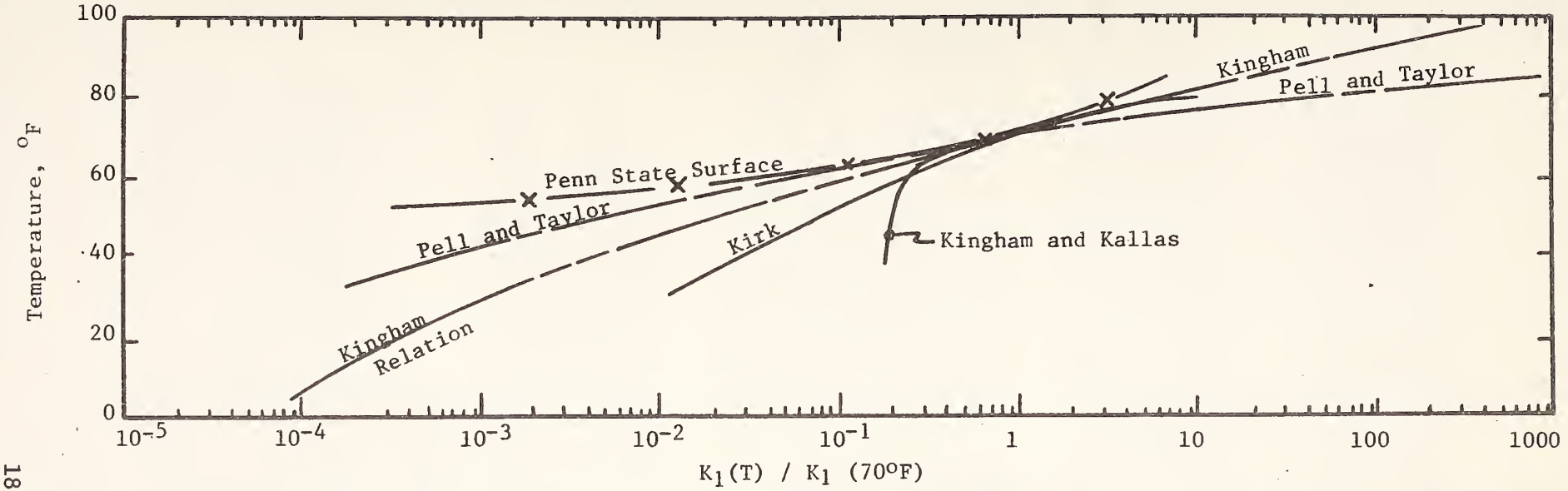
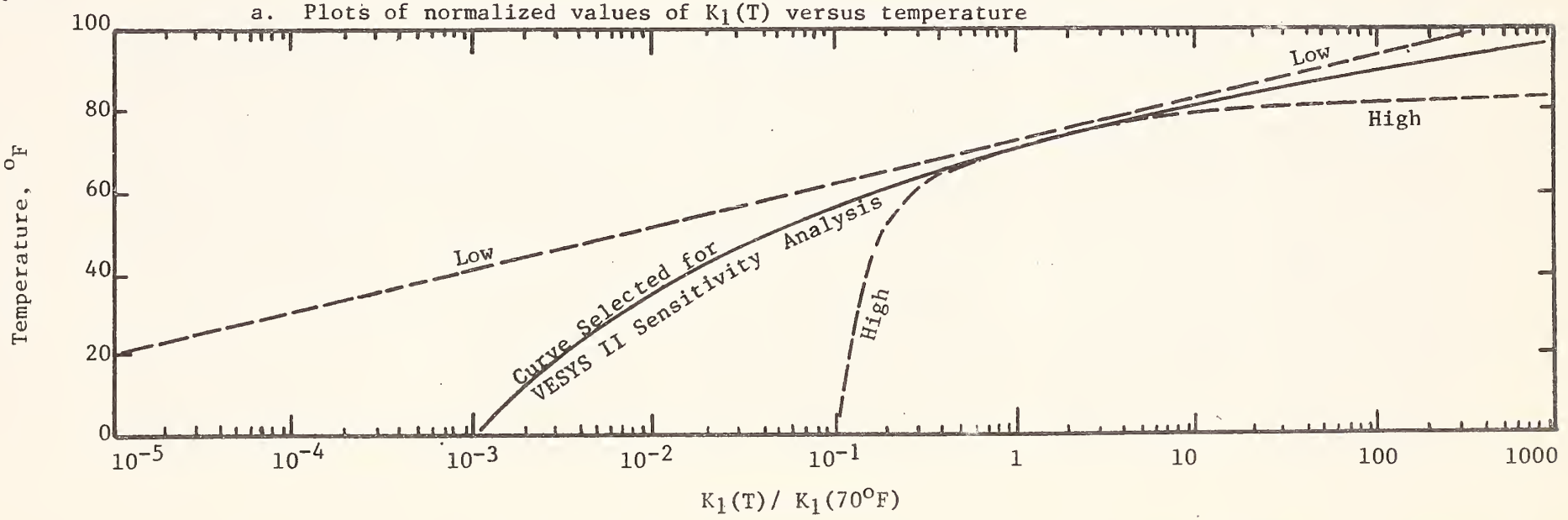


Figure 72. Fatigue exponent K_2 as a function of temperature for different mixes.



a. Plots of normalized values of $K_1(T)$ versus temperature



b. Plots of normalized values of $K_1(T)$ to use for modifying $K_1(70^oF)$ in VESYS II sensitivity analys

Figure 73. Relation between normalized K_1 and temperature

Comparisons indicate similarity in trends for all the data, but fairly gross variations in the degree of reduction in fatigue life as the temperature decreases below the 70°F norm. Somewhat better agreement existed as temperatures increase above the 70°F norm, except that the increase in fatigue life for the Pell and Taylor data were an order of magnitude greater at 80°F and would be several orders of magnitude greater at 100°F.

Envelopes of normalized values appear as dashed lines in Figure 73b identified as producing low and high fatigue life. The solid line shown is the relation selected for use to develop values of $K_1(T)$ for the VESYS IIM sensitivity study. Revised values of $K_1(T)$ are obtained by multiplying $K_1(70°F)$ by the normalized value for the temperature T. The selected relation was drawn in by judgement, giving emphasis to American materials. The Kirk data also falls near this relation.

Figure 74 provides similar normalized plots for $K_2(T)$ versus temperature. As K_2 is the inverse of the absolute value of the slope of the fatigue relation, a constant normalized value of 1.00 implies that the fatigue curves are parallel on their log-log plots. As can be seen, the Kingham and Kallas, Kirk and Kingham relation (See Table 17) data all are approximately constant with temperature, but decreased with increasing temperature when varying at all. Pell and Taylor's $K_2(T)$ decreased markedly with increasing temperature as does the Penn State relation below 60°F. Although limited to two observations each, the asphalt treated base and sand asphalt base tested by Kingham and Kallas (See Table 17) demonstrated parallel lines with temperature changing.

There appears to be a trend in much of the data toward a slight decrease in the exponent K_2 with increasing temperature, although much of the data indicates no variation with temperature. As the value of the exponent makes a considerable difference in fatigue life, it appears more accurate to allow some slight variation in the normalized value such as -0.001 per °F increase in temperature. An equation for this variation to be used for the VESYS II (M) sensitivity analysis is:

$$K_2(T) = K_2(70°F)[1 - 0.001\ (T-70)] \qquad (37)$$

A capability for overriding the VESYS II internal routine for revision of K_1 and K_2 with input values of $K_1(T)$ and $K_2(T)$ has been written into the program. The relations previously described may be utilized to calculate $K_1(T)$ and $K_2(T)$ for each temperature value in the TEMPS array of temperatures once a fatigue curve is selected and $K_1(70)$ and $K_2(70)$ are calculated. It is believed that much more accurate predictions of fatigue life will result. A typical set of fatigue curves developed from use of the relation for $K_1(T)/K_1(70°F)$ from Figure 73b and using Equation (37) appear in Figure 75.

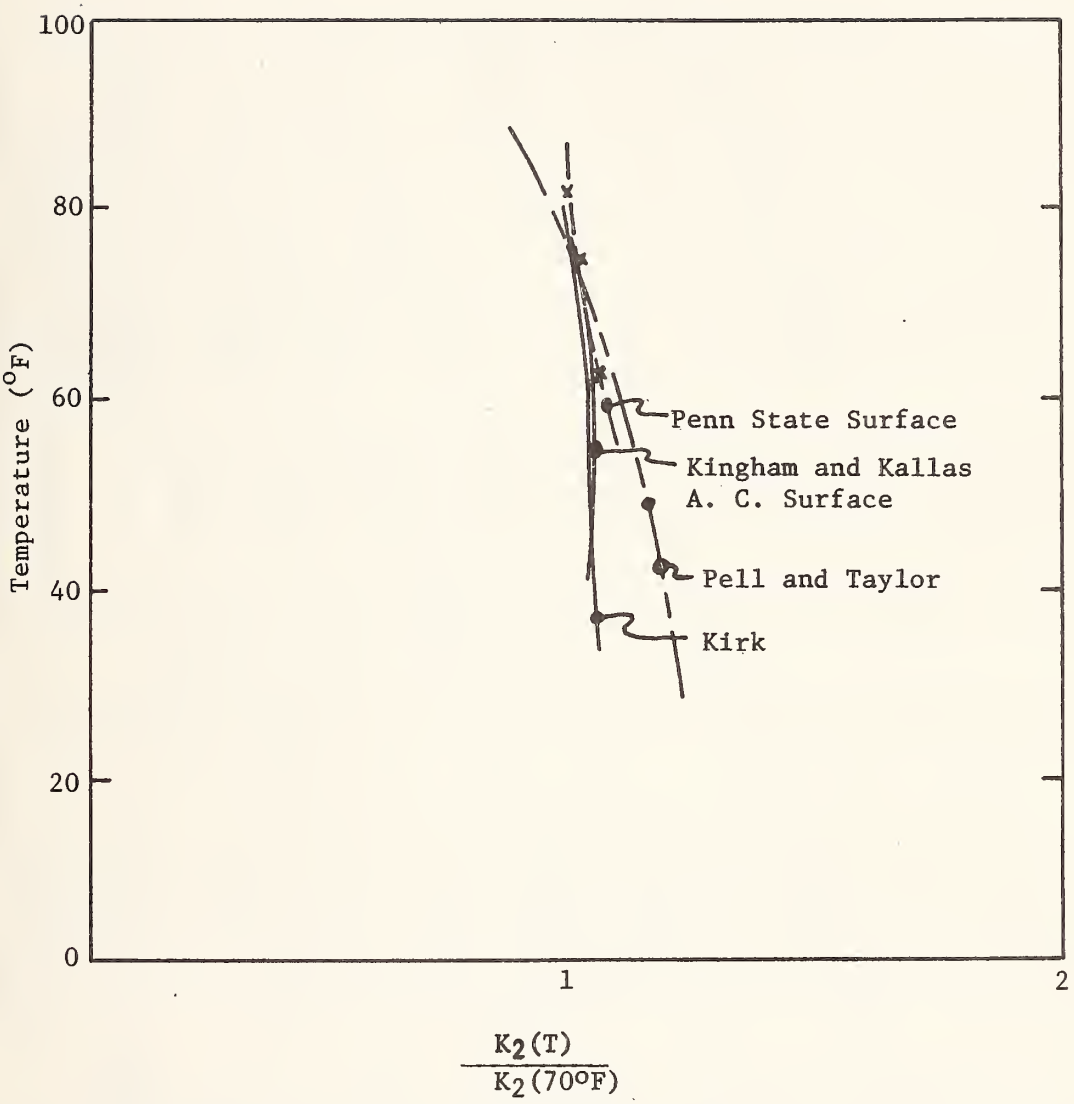


Figure 74. Relation between normalized K_2 and temperature.

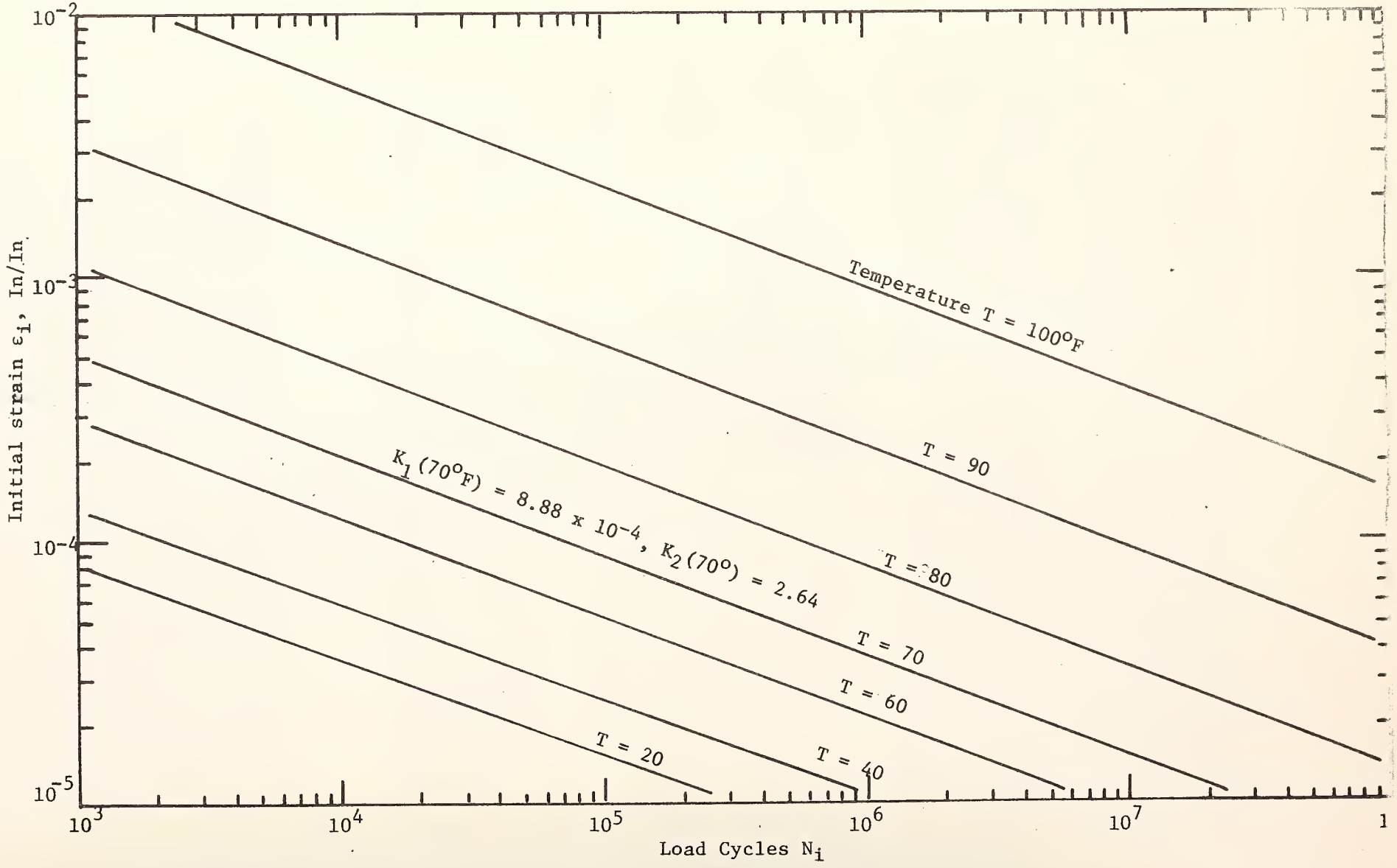


Figure 75. Variation in fatigue life relationship with temperature.

It should be noted that the relations described above also may be used to arrive at $K_1(70)$ and $K_2(70)$ if fatigue curves are available at other test temperatures.

More Rational Fatigue Relationships

Values of K_{1_i} and K_{2_i} have been assessed in the USA by series of uniaxial dynamic beam tests with varying initial strain (or stress) levels, or more recently by biaxial dynamic indirect tensile tests. In other countries, biaxial testing through repetitive torsional loads has been used on specimens of various shapes. These tests give somewhat different results, but share one thing in common. Predictions of fatigue life using test values of K_{1_i} and K_{2_i} generally fall considerably short of actual fatigue lives for the same materials in a pavement slab under service conditions. Although there are several reasons for this, the following are considered to be primary:

1. Failure in laboratory tests follow shortly after cracks appear, whereas failure in the field is generally defined in terms of the amount of cracks propogated to the surface of the pavement long after initial cracking at the bottom of the surface layer.

2. Actual stress and strain conditions in a pavement reticulated by cracks in the bottom are considerably different from those estimated using elastic layer theory.

3. Minor material or test variations in K_{2_i} can cause major variations in fatigue life due to the exponential nature of the relation.

Two approaches have been taken to obtaining values of K_{1_i} and K_{2_i} that relate to a definition of failure at the surface of a pavement under service conditions. One is the statistical relation of number of known load applications experienced at the AASHO Road Test to strains estimated through use of elastic layer theory reported by Kingham (Ref 221), Witczak (Ref 232) and Austin Research Engineers

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[1]Kingham, R. I., "Failure Criteria Developed From AASHO Road Test Data", Proceedings Third International Conference on the Structural Design of Asphalt Pavements, 1972.

[2]Witczak, M. W., "Design of Full-Depth Asphalt Airfield Pavements", Proceedings, Third International Conference on the Structural Design of Asphalt Pavements, 1972.

(Ref 24[1]). The other is use of the so-called Wheel Tracking Test (WTT) by Shell Laboratories (Ref 25[2]) to test heavily-instrumented A.C. slab sections resting on rubber pads and subjected to two-directional wheel loading in a fixed path. These relationships are shown as solid lines in Figure 76 while typical laboratory test results for essentially the same mixes as used in the WTT's appear in dashed lines. Note that the slopes of the WTT and AASHO-based curves are quite different, but the curves approach each other out in the range of greatest interest around 5 to 15 million load applications. The laboratory tests would give much smaller predictions of fatigue life.

Figure 77 from Ref 25 shows typical strain measurements versus number of load cycles for a WTT on the typical Dutch Mix AC-I. Note the stages of crack development at the bottom of the slab and that several times as many loads are required to develop sufficient cracking to represent failure as are required to initiate cracking at the bottom (unlike the standard laboratory tests). Cracks only appeared at the surface near the last stage designated as "failure: in Fig. 77.

It is the opinion of the authors that the fatigue relationships plotted in solid lines in Figure 76 are more rational and may be expected to give more realistic predictions than typical results from standard laboratory tests. Much more such data is needed.

Selection of Fatigue Curves to Represent A Reasonable Range for A. C. Pavements in Service

Based on the rationale in the previous section, it was decided to utilize the fatigue relationships from the Shell Wheel Tracking Tests and the AASHO Road Test data-based developments as source material rather than the laboratory test results expected to give low estimates of fatigue life. Actual selections were based on study and judgement as follows to represent a mean plus and minus one standard deviation:

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[1]Austin Research Engineers Inc., "Development of New Design Criteria for Asphalt Concrete Overlays of Flexible Pavements," Preliminary Report No. FH 2/1, FHWA Contract No. DOT-FH-11-8544, June 1975.

[2]Van Dijk, W., "Practical Fatigue Characterization of Bituminous Mixes", paper prepared for presentation at the 1975 Annual Meeting of the Association of Asphalt Paving Technologists, Phoenix, Arizona, February 1975.

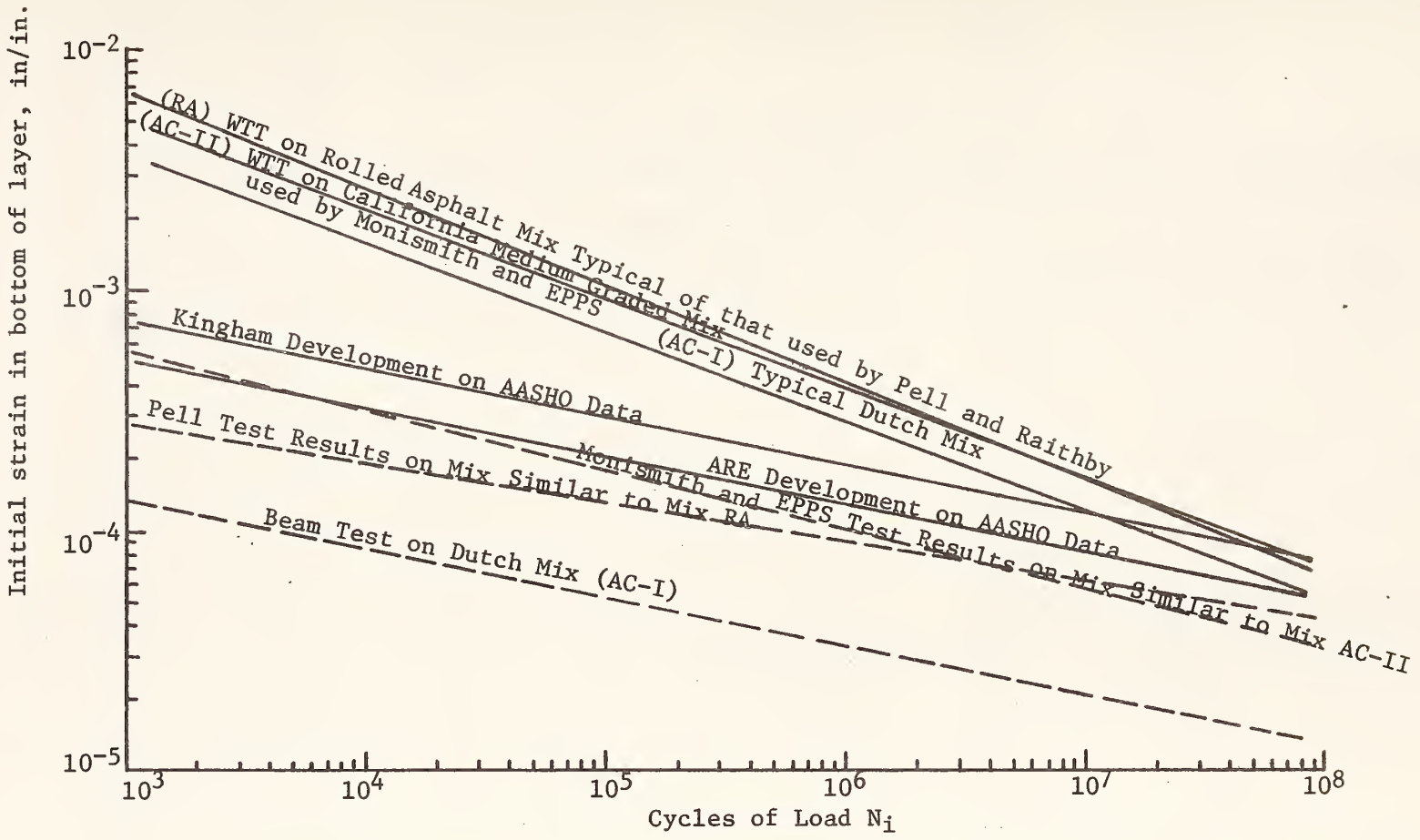

Initial strain in bottom of layer, in/in.
10^{-2}
10^{-3}
10^{-4}
10^{-5}
10^{3}
10^{4}
10^{5}
10^{6}
10^{7}
10^{8}
Cycles of Load N_i
(RA) WTT on Rolled Asphalt Mix Typical of that used by Pell and Raithby
(AC-II) WTT on California Medium Graded Mix used by Monismith and EPPS
(AC-I) Typical Dutch Mix
Kingham Development on AASHO Data
ARE Development on AASHO Data
Monismith and EPPS Test Results on Mix Similar to Mix AC-II
Pell Test Results on Mix Similar to Mix RA
Beam Test on Dutch Mix (AC-I)

Figure 76. Plots for fatigue relationships from Wheel Tracking tests, analytical developments based on AASHO Test data and beam tests.

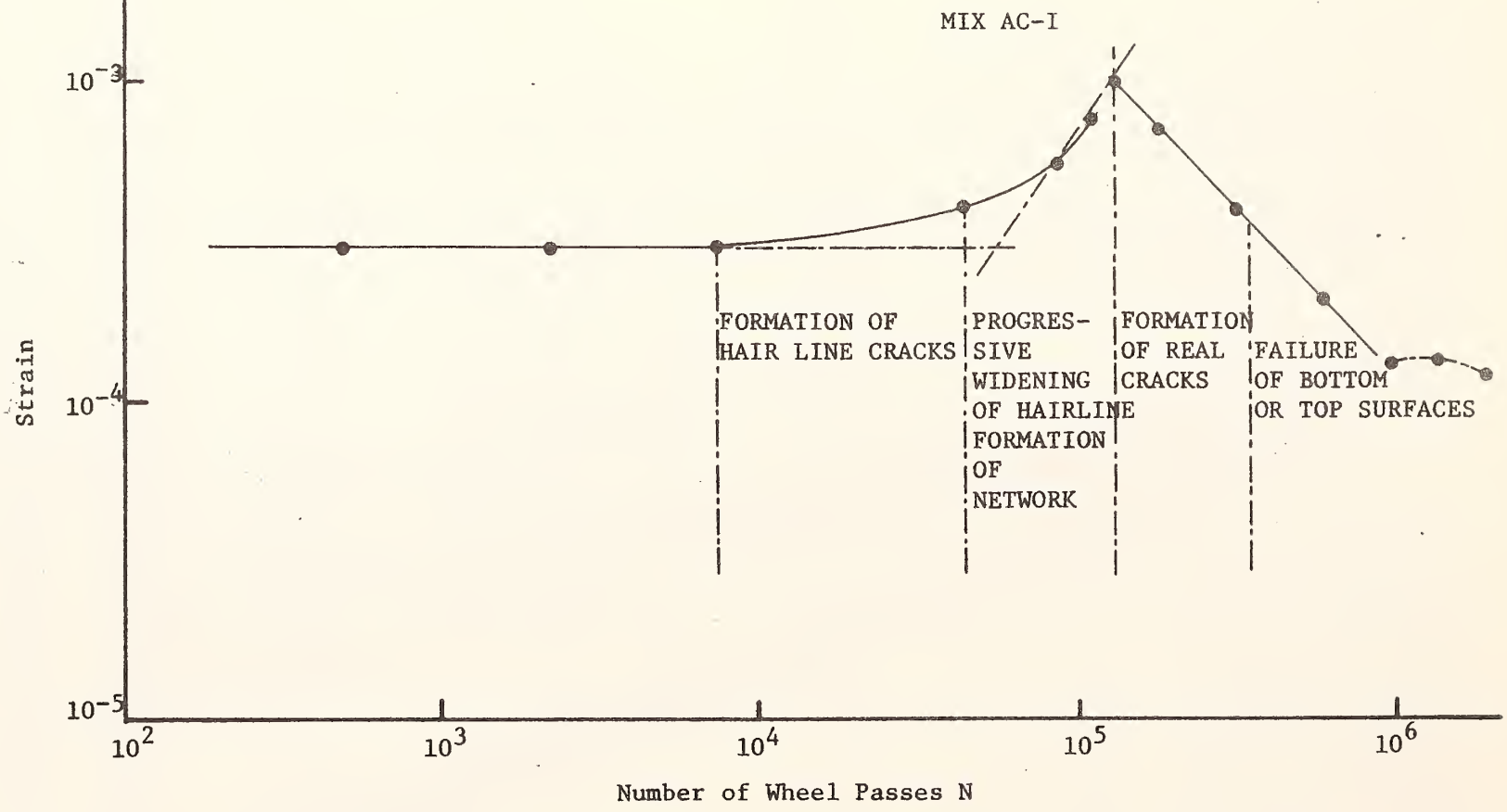


Figure 77. Crack development as a function of strain readings at equivalent number of wheel passes (from Ref 5).

1. The low fatigue potential was selected to approximate the "ARE Development on AASHO Data" in Figure 76.
2. The high level was selected to approximate the "WTT on California Medium Graded Mix" in Figure 76.
3. A mean was established that would approximate an average curve between the low and high curves.

The curves selected for the main sensitivity analysis appear in Figure 78. The values of $K_1(T)$ and $K_2(T)$ developed using the curve in Figure 73 and Equation (37) for different months and temperatures appear in Table 18.

The temperatures were selected on the basis of judgement and are those for Austin, Texas as high and Urbana, Illinois as low. Neither represents the extremes of the range of temperatures in the United States, but appear to represent approximately a standard deviation above and below the mean.

The Fatigue relationship selected for use on the AASHO Road Test predicted performance was that in Figure 76 called "Kingham Development on AASHO Data." This appeared reasonable as it represented actual performance of the materials involved. The temperature range was, of course, that for Urbana, Illinois. The calculated values of $K_1(T)$ and $K_2(T)$ appear in Table 19.

It was necessary also to establish values for the stochastic input variables COEFK1 and COEFK2, which are the coefficients of variation for K_1 and K_2, respectively. The best data available are those reported in a previous section of this Appendix for laboratory test results. No stochastic data really exist for the more rational WTT and AASHO Road Test fatigue relationships. Assuming that the coefficients of variation for K_1 and K_2 from the laboratory test data apply, the "between-project coefficients of variation are 3.15 for K_1 and 0.26 for K_2. As we are concerned with "in-project" variation in VESYS IIM solutions, much reduced numbers must be developed with study, judgement and experimentation as a basis.

It was decided that a mean value of .07 for COEFK2 would be a reasonable expectation for variability in pavements built with the current state-of-the-art construction techniques and with current quality control requirements. A low level of 0.04 and a high level of 0.1 were also selected through judgement. Through experimentation with different possible rotations of the line on the log-log fatigue plot to change the slope such that K_2 varied by 4% to 10%, calculating the resulting values of K_2 and using judgement, a mean value of 0.48 for COEFK1 resulted with a low level of 0.30 and a high level of 1.24. In explanation of the nature of this distribution, it should be noted

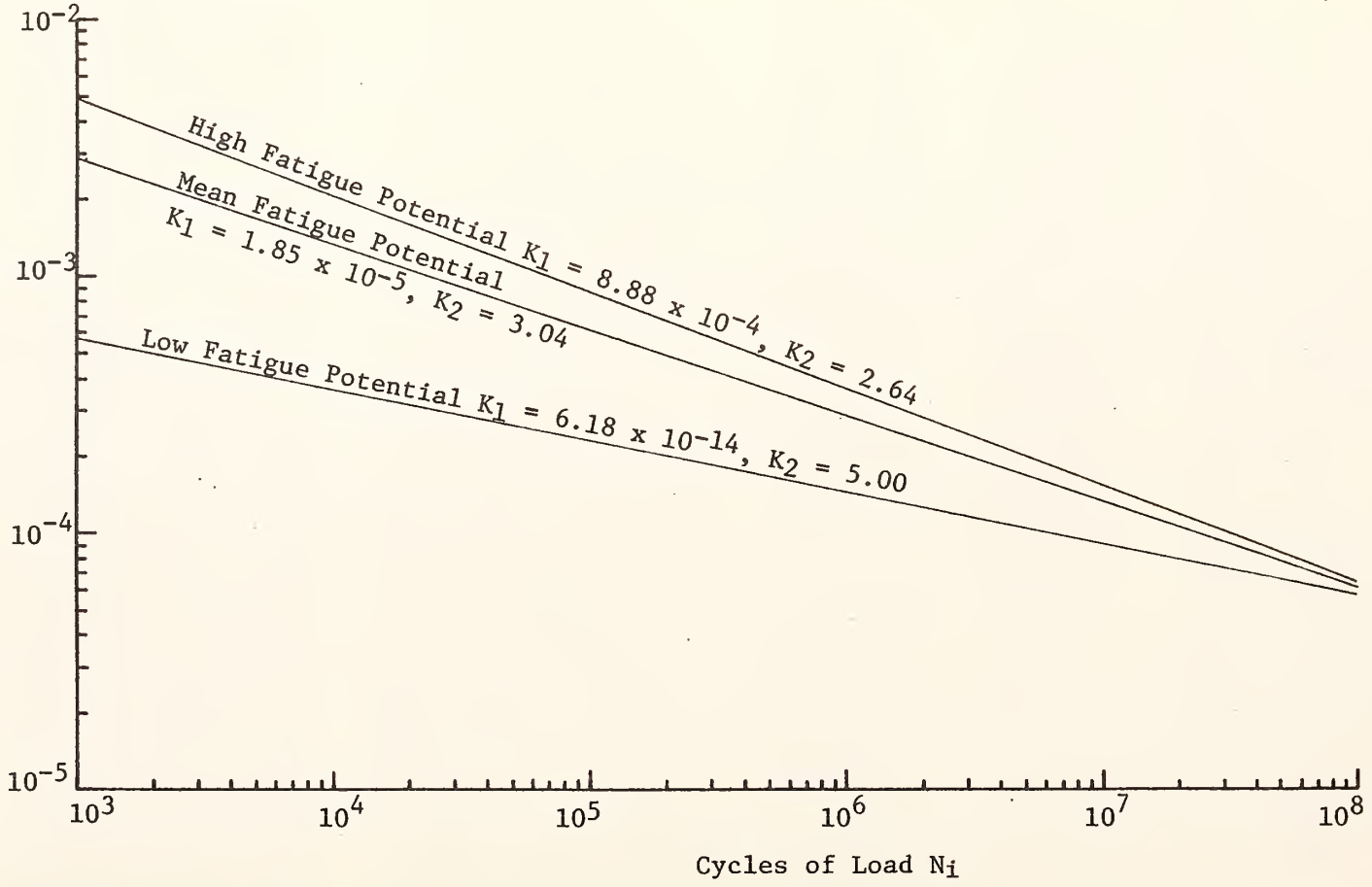


Figure 78. Fatigue relations at 70°F selected for the main sensitivity analysis of VESYS IIM

Table 18. Values for temperatures and fatigue life potential used for the sensitivity analysis.

	Month	Temp	$K_1(T)/K_1(70)$	$K_2(T)/K_2(70)$	Fatigue Life Potential					
					Low		Medium		High	
					K_1	K_2	K_1	K_2	K_1	K_2
TEMPS	LOW J	21.5	.0033	1.049	2.04×10^{-16}	5.25	6.11×10^{-8}	3.19	2.93×10^{-6}	2.77
	F	23.5	.0036	1.046	2.22×10^{-16}	5.23	6.66×10^{-8}	3.18	3.2×10^{-6}	2.76
	M	25.5	.0043	1.044	2.65×10^{-16}	5.22	7.96×10^{-8}	3.17	3.82×10^{-6}	2.76
	A	45.0	.030	1.025	1.85×10^{-15}	5.13	5.55×10^{-7}	3.12	2.65×10^{-5}	2.71
	M	60.5	.22	1.009	1.36×10^{-14}	5.05	4.07×10^{-6}	3.07	1.95×10^{-4}	2.66
	J	69.0	.90	1.001	5.56×10^{-14}	5.01	1.67×10^{-5}	3.04	7.99×10^{-4}	2.64
	J	73.0	1.60	.997	9.89×10^{-14}	4.99	2.96×10^{-5}	3.03	1.42×10^{-3}	2.63
	A	75.0	2.70	.995	1.67×10^{-13}	4.98	5.0×10^{-5}	3.02	2.4×10^{-3}	2.63
	S	71.0	1.20	.999	7.42×10^{-14}	5.0	2.22×10^{-5}	3.04	1.07×10^{-3}	2.64
	O	67.5	.14	1.003	8.65×10^{-15}	5.02	2.59×10^{-6}	3.05	1.24×10^{-4}	2.65
	N	44.5	.03	1.025	1.85×10^{-15}	5.13	5.55×10^{-7}	3.12	2.66×10^{-5}	2.71
	D	39	.015	1.031	9.27×10^{-16}	5.16	2.78×10^{-7}	3.13	1.33×10^{-5}	2.72
	HIGH J	49.7	.052	1.020	3.21×10^{-15}	5.10	9.62×10^{-7}	3.10	4.62×10^{-5}	2.69
	F	53.3	.082	1.017	5.07×10^{-15}	5.09	1.52×10^{-6}	3.09	7.28×10^{-5}	2.68
	M	59.5	.175	1.011	1.08×10^{-14}	5.06	3.24×10^{-6}	3.07	1.55×10^{-4}	2.67

Table 18. Values for temperatures and fatigue life potential used for the sensitivity analysis. (cont.)

Month		Temp	$K_1(T)/K_1(70)$	$K_2(T)/K_2(70)$	Fatigue Life Potential					
					Low		Medium		High	
					K_1	K_2	K_1	K_2	K_1	K_2
TEMPS HIGH	A	68.6	.90	1.001	5.56×10^{-14}	5.01	1.67×10^{-5}	3.04	7.99×10^{-4}	2.64
	M	75.2	2.72	.9948	1.68×10^{-13}	4.97	5.03×10^{-5}	3.02	2.42×10^{-3}	2.63
	J	81.6	14.0	.9884	8.65×10^{-13}	4.94	2.59×10^{-4}	3.0	1.24×10^{-2}	2.61
	J	84.6	32.0	.9854	1.98×10^{-12}	4.93	5.92×10^{-4}	3.0	2.84×10^{-2}	2.60
	A	84.7	33.0	.9853	2.04×10^{-12}	4.93	6.11×10^{-4}	3.0	2.93×10^{-2}	2.60
	S	78.9	6.4	.9911	3.96×10^{-13}	4.96	1.18×10^{-4}	3.01	5.68×10^{-3}	2.62
	O	70.1	1.05	1.000	6.49×10^{-14}	5.00	1.94×10^{-5}	3.04	9.32×10^{-4}	2.64
	N	59.1	.17	1.011	1.05×10^{-14}	5.06	3.15×10^{-6}	3.07	1.51×10^{-4}	2.67
	D	52.3	.074	1.018	4.57×10^{-15}	5.09	1.37×10^{-6}	3.09	6.57×10^{-5}	2.69

Fatigue Life Potential Input Variables at 70°F:

	Level of NFAIL		
	Low	Medium	High
$K_1(70^oF)$	6.18×10^{-14}	1.85×10^{-5}	8.88×10^{-4}
$K_2(70^oF)$	5.00	3.04	2.64

Table 19. Values for temperatures and fatigue life potential coefficients $K_1(T)$ and $K_2(T)$ used for the predictions of performance for AAAHO Road Test sections.

Month	Temperature	$\frac{K_1(T)}{K_1(70^oF)}$	$\frac{K_2(T)}{K_2(70^oF)}$	$K_1(T)$	$K_2(T)$
January	42.2	0.02	1.03	4.74×10^{-15}	5.14
February	28.3	0.006	1.04	1.42×10^{-15}	5.21
March	22.0	0.0038	1.05	9.01×10^{-16}	5.24
April	26.5	0.005	1.04	1.18×10^{-15}	5.22
May	38.0	0.014	1.03	3.32×10^{-15}	5.16
June	52.2	0.061	1.02	1.45×10^{-14}	5.09
July	63.0	0.30	1.01	7.11×10^{-14}	5.04
August	72.4	1.40	0.98	3.32×10^{-13}	4.99
September	75.2	2.70	0.99	6.40×10^{-13}	4.97
October	72.8	1.70	1.00	4.03×10^{-13}	4.99
November	66.1	0.48	1.00	1.14×10^{-13}	5.02
December	56.1	0.101	1.01	2.40×10^{-14}	5.07

Fatigue life potential input variables at 70^oF.

$K_1(70^oF)$ 2.37×10^{-13}

$K_2(70^oF)$ 5.00

that the distribution of K_1 is skewed and would be more nearly log normal than normal.

As the quality control and construction for the AASHO Road Test pavements were of a high level, the low levels of variability were used in the predictions of performance of AASHO Road Test Sections, that is, COEFK1 = 0.48 and COEFK2 = 0.04.

Feasibility Of Using Only "COUPLED" Values for K_1 and K_2

As K_1 and K_2 are strongly negatively correlated (see Figure 69), it appears that irrational fatigue life potential relationships were likely if K_1 and K_2 were viewed as independent variables in the sensitivity analysis. It was decided to study briefly what the effect might be if the values of $K_1(70^oF)$ and $K_2(70^oF)$ selected for the low, mean and high levels for the main sensitivity analysis were allowed to vary freely or remained as coupled or paired variables representing a single characterization.

The factorial for this study is then as follows:

K_2 (70°F) \ K_1 (70°F)	6.18×10^{-14}	1.85×10^{-5}	8.88×10^{-4}
2.64	x	x	x
3.04	x	x	x
5.00	x	x	x

In order to plot these values and see what the curves would look like, calculate the intercept of initial strain at $N_i = 10^3$.

$$\varepsilon(N_i = 1000) = \left(\frac{K_1}{1000}\right)^{1/K_2}$$

K_2 (70°F) \ K_1 (70°F)	6.18×10^{-14}	1.85×10^{-5}	8.88×10^{-4}
2.64	1) 7.25×10^{-7}	2) 1.18×10^{-3}	3) 5.00×10^{-3}
3.04	4) 4.66×10^{-6}	5) 2.80×10^{-3}	6) 1.02×10^{-2}
5.00	7) 5.73×10^{-4}	8) 2.84×10^{-2}	9) 6.16×10^{-2}

The coupled values to be used in the main sensitivity analysis appear on the diagonal in blocks 7, 5, and 3 and the fatigue relations they represent are plotted in Figure 79 in solid lines. It can be seen that the fatigue curves for the combinations of K_1 and K_2 represented by blocks 1 and 4 are so sensitive to strain that they may not be plotted with the usual relations obtained from testing or analysis of AASHO data. The relations for the combinations represented by blocks 6, 8 and 9 all plotted (see dashed lines) above normal test data, while that for Block 2 plotted as somewhat low in fatigue life.

It was not necessary to obtain VESYS IIM solutions as N_i is the only variable when traffic is fixed and stochastic variations are ignored. This would also amount approximately to a constant revision when the stochastic variables are constant. As a tensile strain at 10^{-4} is rather typical, calculation of N_i for $\varepsilon_i = 5 \times 10^{-5}$, 10^{-4} and 5×10^{-4} will represent the range of interest.

Calculate $N_i = K_1 \left(\frac{1}{\varepsilon}\right)^{K_2}$:

K_1 (70°F)		6.18×10^{-14}	1.85×10^{-5}	8.88×10^{-4}
K_2 (70°F)				
ε_i				
5×10^{-5}	2.64	.014	4.19×10^{6}	2.01×10^{8}
	3.04	0.73	2.20×10^{8}	1.06×10^{10}
	5.00	1.98×10^{8}	5.92×10^{16}	2.84×10^{18}
10^{-4}	2.64	.002	6.72×10^{5}	3.22×10^{7}
	3.04	.089	2.67×10^{7}	1.28×10^{9}
	5.00	6.18×10^{6}	1.85×10^{15}	8.88×10^{16}
5×10^{-4}	2.64	.00003	9592	4.60×10^{5}
	3.04	.0007	2.00×10^{5}	9.63×10^{6}
	5.00	1978	5.92×10^{11}	2.84×10^{13}

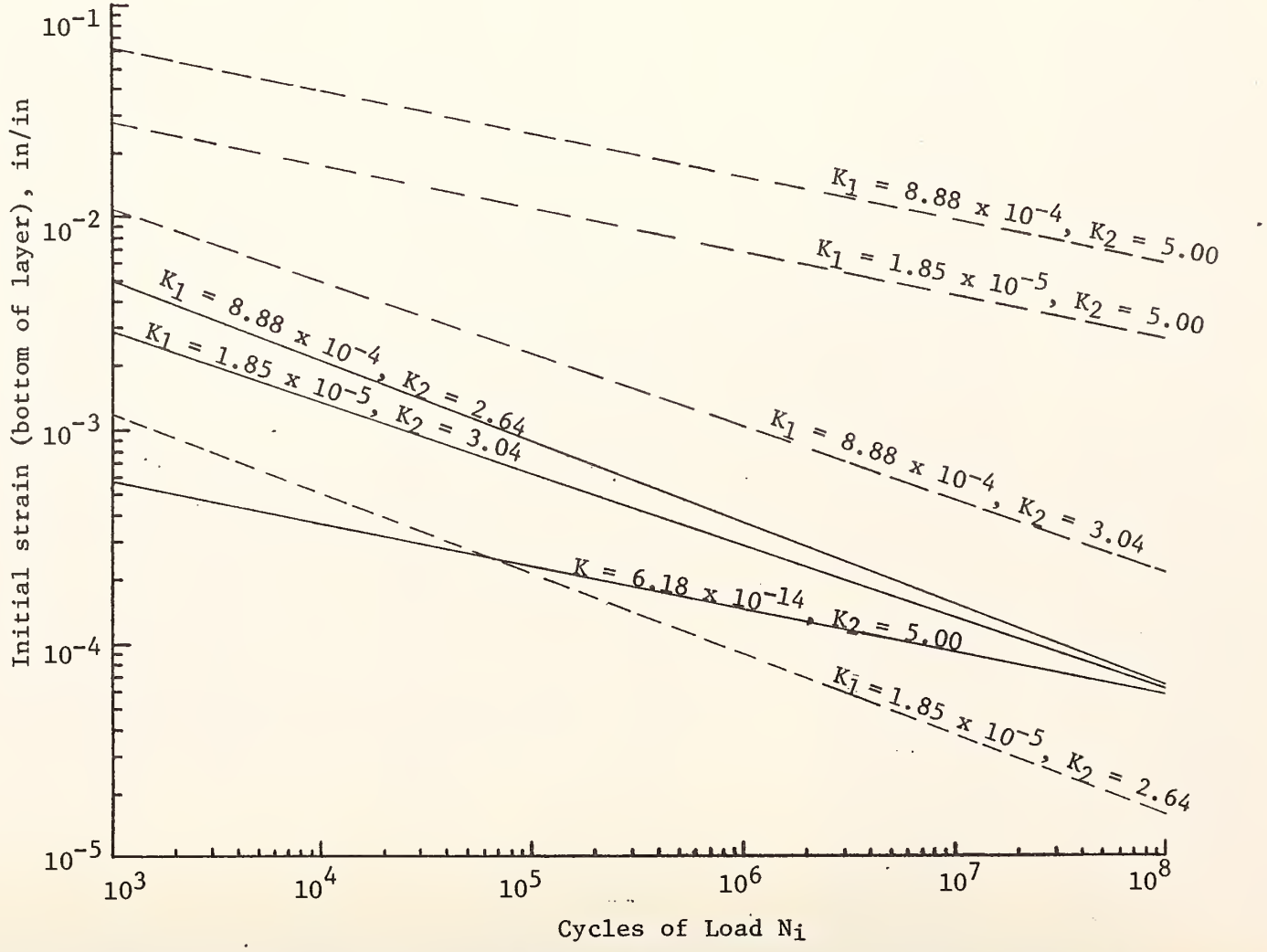


Figure 79. Plots of fatigue curves for small factorial with K_1 and K_2 uncoupled.

The results of the calculations are plotted in Figure 80 for an initial strain level of 10^{-4} in/in. The trends for each strain level were similar so only this level was plotted. The plots for the coupled relations are also included for contrast.

As would be expected from the equation:

$$N_i = K_1 \left(\frac{1}{\varepsilon}\right)^{K_2}$$

the number of loads to failure increase exponentially with K_2 when K_1 and ε_1 are held constant and linearly with K_1 and K_2 and ε_i are held constant.

On the basis of judgement, realistic asphaltic concrete mixes at an initial strain level of 10^{-4} in/in would have a service life of 500,000 to 10^8 cycles of loading at 70°F. Plotting this range on Figure 80a indicates that a large value of K_1 such as 8.88×10^{-4} would only be consistent with values of K_2 between about 2.15 to 2.71. Similarly, a low value of K_1 such as 6.18×10^{-14} is only consistent with values of K_2 between 4.75 and 5.28. This indicates a rather strong negative correlation.

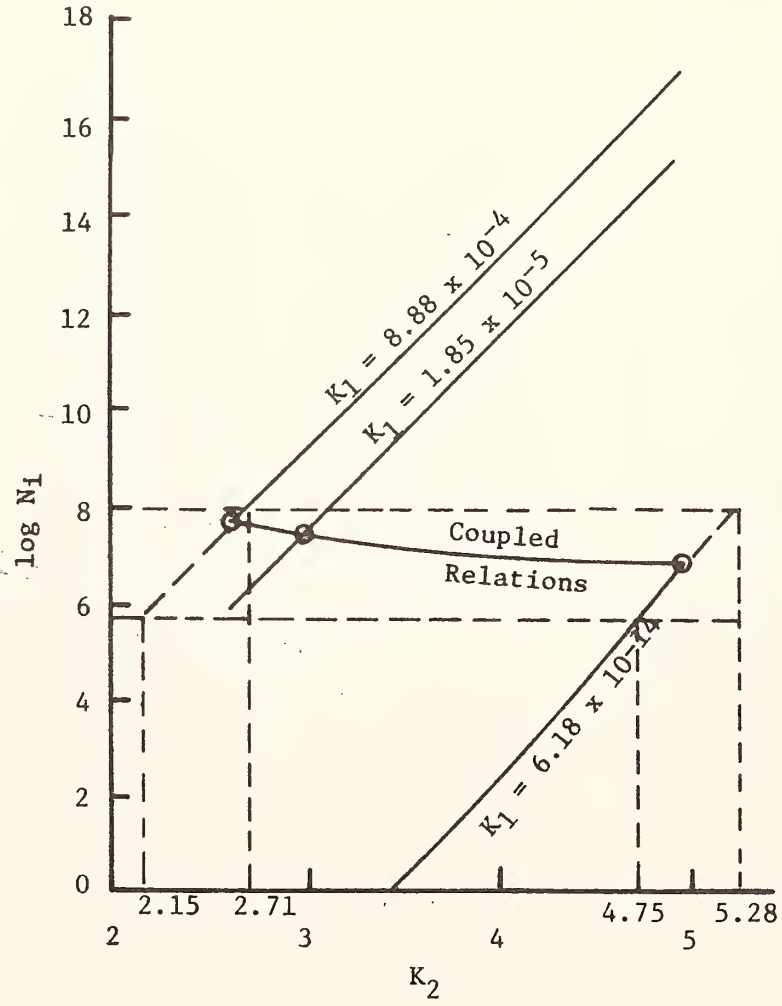
Figure 81 is a plot of K_1 vs. K_2 showing as expected for the coupled values that they are linearly correlated on semi-log paper. Manipulation of the basic fatigue life equation will yield

$$K_2 = \log K_1 \left(\frac{1}{\log \varepsilon}\right) - \frac{\log N_i}{\log \varepsilon},$$

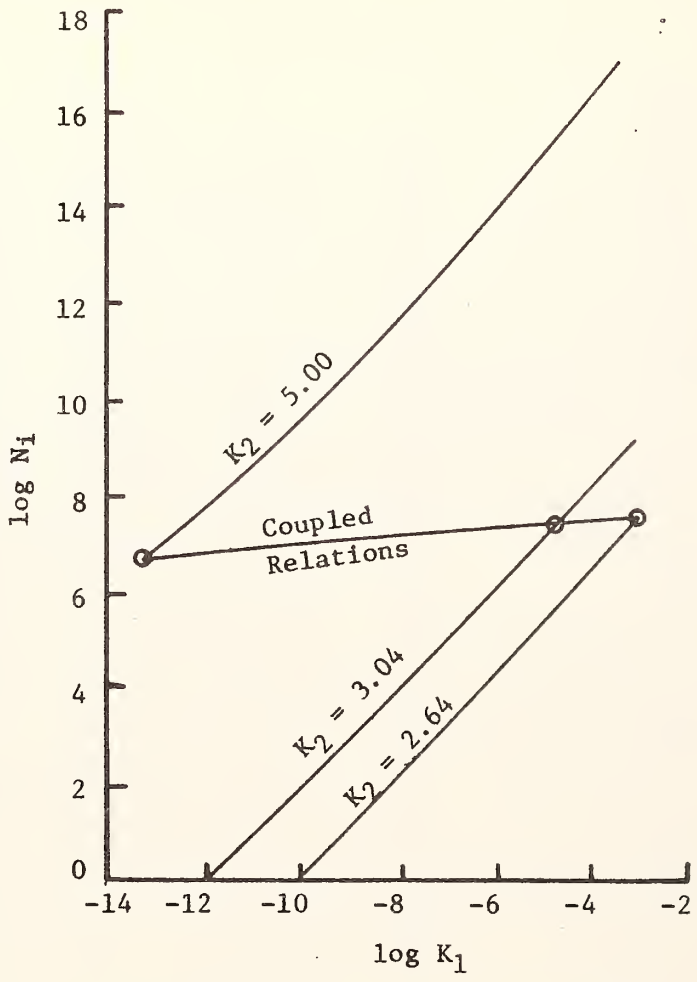
which is a straight line. The uncoupled values are also shown to emphasize their scatter.

Figure 82 is a plot showing results from testing by Pell applying torsion to cylinders, Monismith's beam tests, Shell's Wheel Tracking Tests, Kingham development of AASHO data, ARE's development of AASHO data and the data developed for the VESYS IIM Sensitivity Analysis. It can be seen that all of this data from a variety of field and tests sources and conditions is highly negatively correlated. A correlation coefficient of -0.867 was developed for input as K_1K_2CORL to VESYS IIM to reflect this correlation.

It can be seen from the unusual values of fatigue life in Figure 79, lack of correlation indicated in Figure 80 for the uncoupled values of K_1 and K_2 and the close correlation for real data in Figure 81 that little if any useful data is lost by coupling K_1 and K_2 in the sensitivity analysis. In fact, this was considered a necessity to avoid masking the importance of other variables through unrealistic responses to be expected from the uncoupled values as demonstrated above.



a. Log N_i versus K_2



b. Log N_i versus log K_1

Figure 80. Plots of load cycles calculated versus "uncoupled" values of K_1 and $K_2(\varepsilon_i = 10^{-4})$.

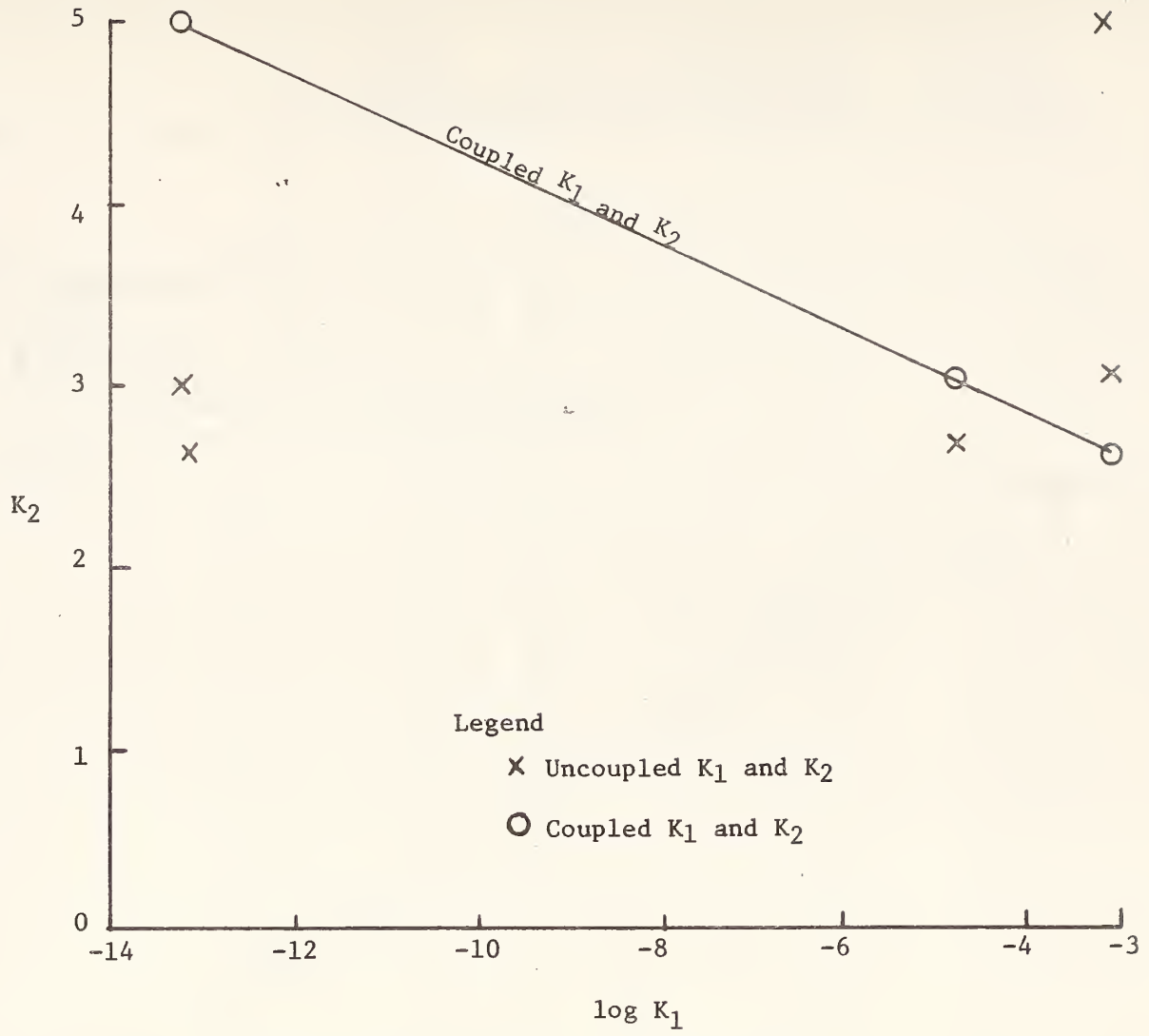


Figure 81. Plot of K_2 versus log K_1

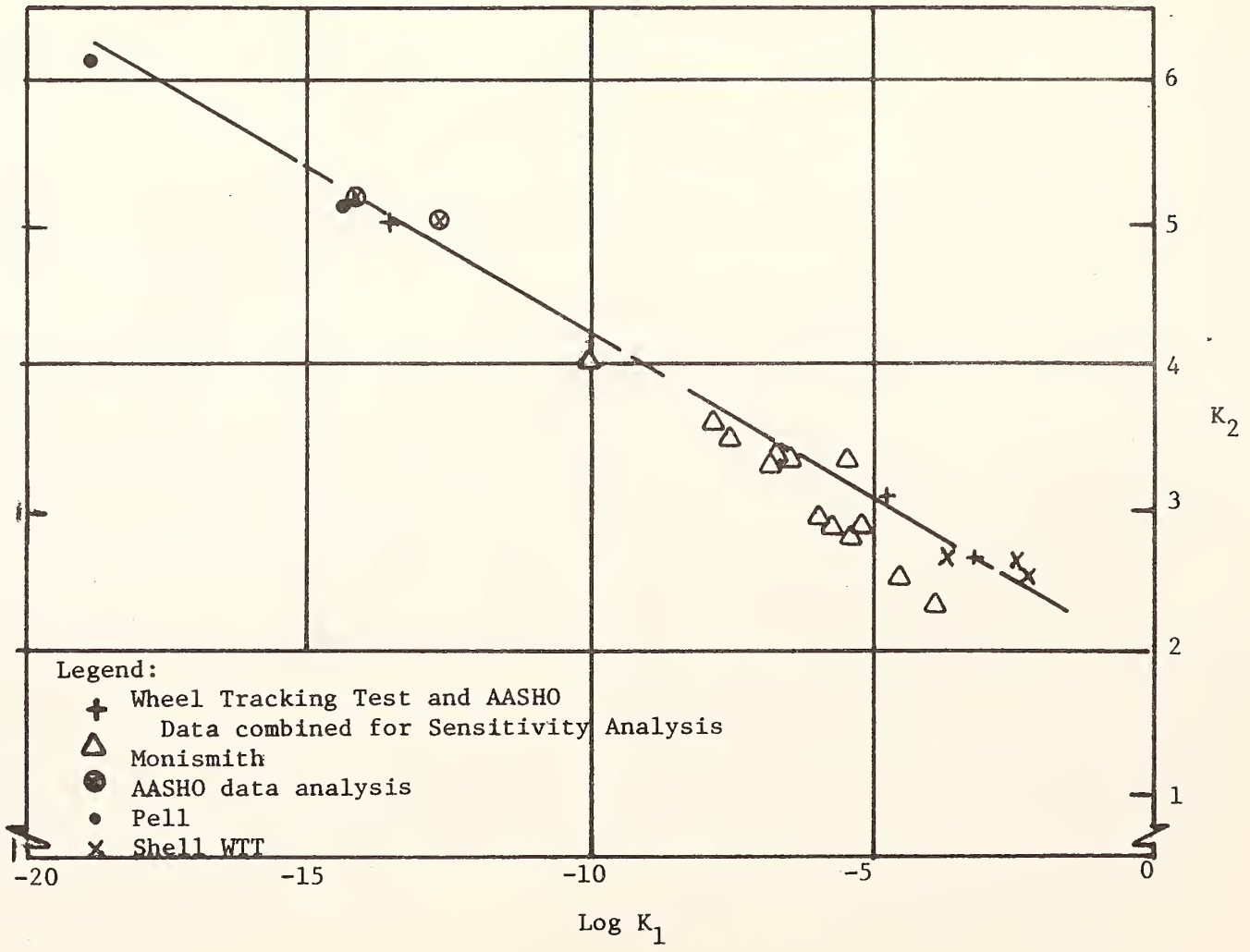


Figure 82. Plots for K_2 versus Log K_1 for beam, wheel tracking, torsional tests and AASHO field measurements.

APPENDIX E

VESYS IIM

INPUT DICTIONARY

This appendix is a reference manual for the VESYS IIM computer program. As all of the inputs to the program are associated with some keyword, it has been found convenient to arrange the main body of the text as a dictionary. Information on each input variable can be found by looking up the associated keyword.

A word of explanation is necessary concerning the concept of "default" settings which is used in the program. Many of the input variables have predefined values. The default value will be used if no value has been supplied for a particular variable. A value can be supplied either by direct input or by being calculated as in intermediate step in a Type 1 analysis (the variables so affected are: GNØRMAL, GRADIAL, CØEFGNRM, CØEFGRAD, ALFNØRM, NDELTAS and DELTAS). It is useful to think of each run of a job as having the values used in the proceeding run by default. Any variables which are to be changed can be input. All others will implicitly remain the same. The default settings listed in this manual will be correct for the first run, subsequent runs will be affected by those preceeding them.

Those who are unfamiliar with the program are referred to Chapter 3 of Structural Analysis Design Guide for Flexible Pavements (Ref 26[1]) for general instruction in its use. Those interested in general information are also referred to the dictionary entry for TYPE, the overall control variable for the program.

The following is a list of all keywords recognized by the VESYS IIM Program. (next page):

---

[1]Kenis, W. J., "Structural Analysis Design Guide for Flexible Pavements", Office of Research, Federal Highway Administration, September 1974 (Unpublished working document).

ALPHA
ALFNØRM***
ALFRAD*
AMPLITUD
BETA
CØEFGNRM***
CØEFGRAD***
CØEFK1
CØEFK2
CØRLCØEF
CØRLEXP
CREEP1,2,3
DEBUG
DELTAS***
DURATIØN
ENDØFJØB
ENDØFRUN
GNØRMAL***
GNU
GNUNØRM***
GNURAD*
GRADIAL***
INDEX
ITYPES
K1K2CØRL
LABDATA
LAMBDA
LAYER1,2,3
LØADING

NDELTAS***
NØPART
NRPØINTS
NTCURVE
NTEMPS
NTRANDØM
NTSTATIC
PSIFAIL
PUNCH
QUALITY0
RADUIS
REFTEMP
RPØINTS
STDEVO
STRNCØEF
STRNEXP
TCURVE
TEMPS
THICK1,2
TITLE**
TØLERNCE**
TSTATIC
TRANDØM
TUNITS
TYPE
UNLØAD
VARCOEF1, 2, 3
VCAMP
VCDUR
ZPØINT

* Recognized but no longer used

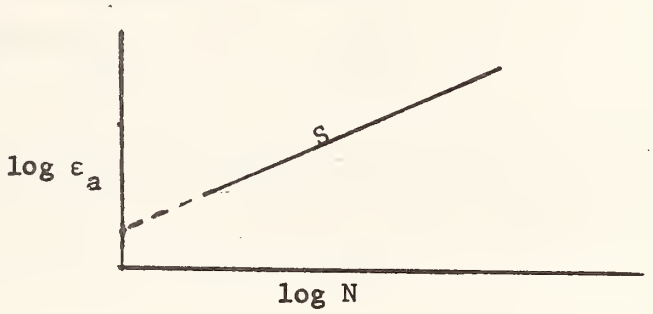
** Added at ARE, not found in FHWA version

*** Calculated as an intermediate step in a Type 1 analysis

ALPHA

Array of permanent deformation materials property parameters for the three layers (see also GNU). The n^{th} element of this array is the value obtained for layer n.

These parameters are determined from repeated load tests on samples of the materials in the layers. The log of the accumulated permanent strain is plotted against the log of the number of load applications. This curve is approximated by a straight line:



where

ε_a is accumulated permanent strain,

N is the number of repetitions, and

S is the slope

The value of ALPHA for the material in this test is defined to be:

$$\alpha = 1-S$$

Tests must be run on the materials for each layer to determine values for ALPHA and GNU (see relevant commentary). The statis load subprogram will use these values to calculate ALFNØRM and GNUNØRM . These two variables form the basis for predicting rutting and slope variancein the repeated load analysis (see relevant commentaries for details).
[Internal array is ALPH(I)]

ALFØRM

The exponent, α, in the system's permanent vertical deformation expression which is used to predict rutting:

$$d_N = d_1 \mu N^{\alpha}$$

where

N is the number of load applications,

d_N is the vertical deformation after N applications,

μ is GNUNØRM, (not the same as GNU), and

α is ALFNØRM. (not the same as ALPHA).

This is one of the parameters passed from the static load subroutine to the repeated load subroutine in a Type 1 run. It need not be input by a user of the program (see TYPE for details). [Internal name is ALF(4)]

ALFRAD

Formerly a part of the system's permanent radial strain expression, this variable is no longer used in VESYS II (M).

AMPLITUD (Default is 80.0)

The mean intensity of repeated loadings in psi. This variable is treated internally as tire pressure, but a user of the program must understand that it has somewhat different physical significance. In modeling the traffic conditions to which a road will be subjected, we need to represent a wide range of wheel weights. In Chapter 4 of Ref 26. Mr. Kenis outlines a good method for obtaining a mean axle weight which is adjusted to account for the effects of tandem axles. AMPLITUD can be computed as follows:

$$\text{AMPLITUD} = \frac{1000 \cdot W}{2} \cdot \frac{1}{\pi r^2}$$

where

W is the mean axle weight in kips,

and r is RADIUS (see relevant commentary).

Only the heavier traffic is considered in determining the mean axle weight, light vehicles should be ignored.

The value input for AMPLITUD will be multiplied by the value of LØADING for which the static response curves were computed. For this reason it is advisable to input 1.0 for LØADING. Then the technique

described above can be applied with out alteration. If some other value has been used for LØADING, then the value of AMPLITUD calculated above should be divided by LØADING.
[Internal name is AMP]

BETA (Default is .162)

This parameter describes the variation in the time-temperature shift factor as a function of temperature. The following relationship is assumed to hold:

$$\log a_T \simeq \beta(T_o - T)$$

where

a_T is the time-temperature shift factor for a temperature T,

T_o is REFTEMP, the reference temperature, and

β is the value input for BETA.

The time-temperature shift factor, a_T, is defined as follows:

$$a_T = \frac{t_T}{t_{T_o}}$$

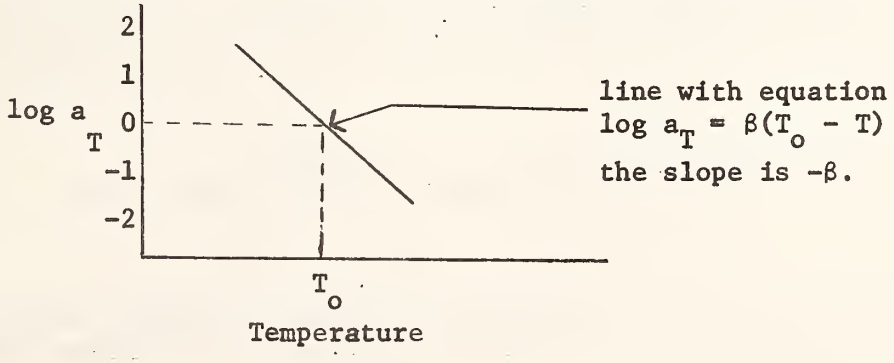
where

t_T is the time required for a given response at temperature T, and

t_{T_o} is the time required for the same response at the reference temperature.

These a_T's are the shift factors which are used to construct the master creep compliance curves LAYER1, LAYER2 and LAYER3 from laboratory tests ar various temperatures (see LAYER commentary).

A plot of log a_T versus temperature approximates a straight line:



According to A Flexible Pavement Analysis Subsystem by (Ref 5)[1] by Kenis and McMahon (page 9) a value of .162 for BETA is "reasonably valid for a wide variety of asphalt mixes."

The value of BETA is applied only to the top layer. It is assumed that the base layer is rock and thus not temperature dependent.
[Internal name is BETA]

CØEFGNRM

Coefficient of variation for the GNØRMAL array. This needs not be input unless GNØRMAL is being input (ordinarily not necessary). This variable should not be set to zero because of its effect on the variance of rut depth which is the basis of the estimate for slope variance, a fundamental parameter in the AASHO equation which is used to predict serviceability:

$$PSI = 5.03 - 1.91 \log_{10}(10 + SV) - .01\sqrt{C + P} - 1.38\ (RD)^2$$

where

PSI is present serviceability index,

SV is slope variance in radians x 10^6,

C is area of cracking in square yards per 1000 sq. yds.,

P is area of patching in square yards per 1000 sq. yds., and

RD is rut depth in inches.

The program actually uses a stochastic form of this equation which treats SV, C + P, and RD as random variables and solves for the expected value of PSI:

$$E[PSI] = 5.03 - 1.91\ E[\log_{10}(1 + SV)] - .01\ E]\ C + P - 1.38\ E[RD^2]$$

The individual stochastic terms in this equation can be approximated by the expected value of the second-order Taylor series about the mean for the log, square root, and square functions. For a general function, f, we can write:

$$f(x) \simeq f(\bar{x}) + (x - \bar{x})\ f'(\bar{x}) + \frac{(x - \bar{x})^2}{2}\ f''(\bar{x})$$

---

[1] Kenis, W. J. and Dr. T. F. McMahon, "A Flexible Pavement Analysis Subsystem," Paper presented at 53rd Annual Meeting of Highway Research Board, January 1974.

Taking the expected value yields:

$$E[f(x)] \simeq f(\bar{x}) + E[x - \bar{x}]\ f'(x) + \frac{E\ (x - \bar{x})^2}{2}\ f''(\bar{x})$$

but

$$E[x - \bar{x}] = 0, \text{ since } E[x] = \bar{x}$$

and

$$E[(x - x)^{-2}] = \text{Var}[x] \text{(The variance of x)}$$

hence

$$E[f(x)] \simeq f\ (\bar{x}) + \frac{\text{Var}[x]}{2}\ f''(\bar{x})$$

From this we compute:

$$E[\log_{10}(1 + SV)] \simeq \log_{10}\ (1 + E[SV] + \frac{\text{Var}[SV]}{2} - \left[\frac{\log_{10} e}{(1 + E[SV])^2}\right]$$

$$E[RD^2] \simeq E[RD]^2 + \frac{\text{Var}[RD]}{2} \cdot 2$$

$$E[Y^{\frac{1}{2}}] \simeq E[Y]^{\frac{1}{2}} - \frac{1}{8} \cdot \text{Var}[Y] \cdot E[Y]^{-\frac{3}{2}}$$

where:

E[SV]	is mean slope variance
Var[SV]	is variance pf slope variance,
E[RD]	is mean rut depth,
Var[RD]	is variance of rut depth,
E[Y]	is E[C]+ E[P], mean area of cracking plus mean area of patching (Per 1000 sq. ft.), and
Var[Y]	is the variance of Y = C + F.

Note that C + P is treated as a single random variable. In VESYS IIM as it currently stands, E[P]and Var[Y]are set to zero. In effect this means that:

$$E[Y^{\frac{1}{2}}] \simeq E[C]^{\frac{1}{2}}$$

Our overall serviceability equation thus reads:

$$E[PSI] \approx 5.03 - 1.91 \left[\log_{10}(1 + E[SV] - \frac{Var[SV] \cdot \log_{10} e}{2(1 + E[SV])^2} \right]$$

$$-.01 \; E[C]^{\frac{1}{2}} - 1.38[E[RD]^2 + Var[RD]]$$

In this formulation the values of Var[RD], E[SV], and Var[SV| are directly related to the value of CØEFGNRM. It is therefore recommended that great caution be excercised if this variable is to be input by the user.
[Internal name is GCVAR(4)]

CØEFGRAD

Coefficient of variation for the GRADIAL array. This need not be input unless GRADIAL is being input (ordinarily not necessary). It is preferable not to set this variable to zero, because that would distort the results of the stochastic equations in the cracking model.
[Internal name is GCVAR(7)]

CØEFK1 (Default is 0.0)

Coefficient of variation for STRNCØEF (see relevant commentary).
[Internal name is CK1]

COEFK2 (Defalut is 0.0)

Coefficient of variation for STRNEXP (see relevant commentary).
[Internal name is CK2]

CØRLCØEF (Default is 0.0)

The coefficient, B a materials variable in the system's spacial auto-correlation function:

$$\rho(x,y) = A + Be^{-\left[\frac{x - y}{C}\right]^2}$$

If x and y are distances along the wheelpath from some origin then $\rho(x,y)$ is the correlation between the road elevations at those two points.

It is convenient to assume that if x and y are infinitely far apart the correlation will be zero.
Since

$$\lim_{(x-y)\to\infty} \rho(x,y) = A$$

we set A to zero.

If x = y, then we get the correlation of an arbitrary road elevation with itself, which is 1. Hence:

$$1 = \rho(x,x) = A + Be^0 = A + B$$

Since we are assuming A = 0, we have B = 1. For this reason it makes sense to always input a value of one for CØRLCØEF.
[Internal name is B]

CØRLEXP (Default is 1.0)

The exponent, C, a materials variable, in the system's spacial autocorrelation function:

$$\rho(x,y) = A + Be^{-\left[\frac{x-y}{C}\right]^2}$$

For considerations outlined in the commentary on CØRLCØEF it is appropriate to assume that A is zero and B (CØRLCØEF) is one.

This function is not used directly in the program. The second partial derivitive of ρ with respect to x evaluated when x = y is used to get an estimate for slope variance in terms of the variance of rut depth:

$$\frac{\partial^2\rho}{\partial x^2}(x, x) = -\frac{2B}{C^2}$$

This value is found in the program in the following form:

$$S = \frac{2B}{C^2}\,\mathrm{Var[RD]}$$

where

S is slope variance in radians x 10^6,

B is CØRLCØEF (which can be set to 1),

C is CØRLEXP, and

Var[RD] is the variance of rut depth in inches squared.

We can treat this as a regression equation which predicts slope variance in terms of variance of rut depth:

$$S = K \cdot Var[RD]$$

where:

K is a regression coefficient.

Using data for slope variance and rut depth variance we can compute K for some given road whose materials properties are similar to those which we wish to model. Then, setting B to 1 we may solve for C in terms of K:

$$K = \frac{2B}{C^2} = \frac{2}{C^2}$$

hence:

$$C = \left(\frac{2}{K}\right)^{1/2}$$

We may safely assume that K will be positive, slope variance tends to be positively correlated with rut depth variance.
[Internal name is C]

CREEP1

CREEP2

CREEP3

These are arrays of curve-fitted coefficients for Dirichlet series which approximate the creep compliance functions for the corresponding layers:

$$D_t = \sum_{i=1}^{n} C_i e^{-\delta_i t}$$

where:

D_t is the value of creep compliance in some layer at time t,

δ_i is the array of DELTAS,

C_i is the CREEP array for the correct layer, and

n is NDELTAS, the number of elements in the DELTAS and CREEP arrays.

NDELTAS must be input before the CREEP and DELTAS arrays, a maximum of 20 points are permitted.

If curve-fitted creep coefficients are to be input, all three layers should be specified in this manner.

Because the program checks the LAYER data for errors, it is necessary to input some values in these arrays even though they will not be used in the actual calculations. Since curve-fitting the LAYER data does not consume much computer time, it is recommended that creep compliance always be input as a master curve (in LAYER1, LAYER2 and LAYER3) and not as curve-fitted coefficients (in CREEP1, CREEP2 and CREEP3).

If these coefficients are to be read in, it is necessary to input the DELTAS as well. Caution must be excercised in selecting appropriate values for this array (see commentary on DELTAS).
[Internally they are stored in E(I,1), E(I,2), and E(I,3)]

DEBUG

This keyword specified that large numbers of intermediate print-outs are needed for finding an error. This will result in an excessive amount of output, it should be avoided unless absolutely necessary.
[Internal name is NODBUG]

DELTAS

The array of δ_i's used in the Dirichlet series which are used to approximate the creep compliance and system response curves. These functions are approximated by finding G_i's in the following expression which give a least-squares best fit:

$$F(t) = \sum_{i=1}^{n} G_i e^{-\delta_i t}$$

where

f is the function being approximated

t is the independent variable, time

G_i is the array of curve-fitted coefficients,

δ_i is the array of DELTAS, and

n is NDELTAS, the number of elements in the δ and G arrays.

NDELTAS must be input before DELTAS, a maximum of 20 are allowed.
Ordinarily a usaer of the program will not need to input any of these curve-fitted coefficients or DELTAS, The needed information is

passed from the static load subroutine to the repeated load subroutine in a Type 1 analysis. Subsequent Type 3 runs may continue to access these data as desired (see commentary on TYPE for details). If DELTAS are input for a Type 3 run, care must be taken that they are the ones for which the coefficient arrays were computed.

Sometimes the DELTAS selected by the program provide a poor fit. In such cases the user may want to input the DELTAS to be used. If this is done in a Type 1, 2 or 4 run, then the values provided will be used throughout the rest of the job. Inputting DELTAS for a Type 3 run will not affect subsequent Type 1, 2 or 4 runs, the program will select a new set in such cases. Suggestions for selecting DELTAS which will provide a good fit are found in Chapter IV of this volume. The points in the DELTA array should not be too close in value, an order of magnitude difference for successive values is recommended. The last δ is zero, corresponding to the constant coefficient which the function asympototically approaches as t increases without limit.
[Internal array is DELTA(1)]

DURATION (Default is .05)

The mean duration of repeated loadings in the random load analysis. In Synthesis for Rational Design of Flexible Pavements, Part I, (Ref 1)[1], page 38, Moavenzadeh et al, suggest that DURATIØN be approximated by the time taken to transverse three times the diameter of the loaded area. This represents an estimate of the distance over which a vehicle's weight has significant effect. No research has been done to establish a definite value for this distance, the estimate given is open to question. If we accept this estimate as reasonable, we may express the duration of a single load as a function of the speed of the vehicle and the radius of the loaded are:

$$t(r,s) = \frac{2r(\text{number of diameters})}{17.6\ s} = \frac{6r}{17.6\ s}$$

where

r is the RADIUS in inches,

s is the vehicle speed in mph, and

t(r,s) is the duration in seconds.

Treating r and s as random variables we may estimate the mean duration by approximating the function t with a second-order Taylor series about the means of r and s and taking the expected value of this approximating function:

[1]Moavenzadeh, F., Soussou, J.E., Findakly, H. K., "Synthesis for Rational Design of Flexible Pavement", Part I, School of Engineering, Massachusetts Institute of Technology, Cambridge, Mass., 02139, January 1973.

$$E[t(r,s)]) \approx t(E[r],E[S] + \frac{\partial^2 t(E[r],E[s])}{\partial r^2} \cdot \frac{Var[r]}{2} +$$

$$\frac{\partial^2 t(E[r],E[s]}{\partial r \partial s} Cov[r,s] + \frac{\partial^2 t(E[r],E[s]}{\partial^2} \cdot \frac{Var[s]}{2}$$

where

E[r] is the mean radius,

Var[r] is the variance of radius

E[s] is the mean speed,

Var[s] is the variance of speed, and

Cov[r;s] is the covariance of radius and speed.

We assume that there is no correlation between a vehicle's speed and the size of its tire print, hence the covariance of r and s will be zero. We also note that the second partial derivitive of t with respect to r is zero. Hence we have:

$$DURATION = E[t(r,s)] \approx t(E[r],E[s]) + \frac{\partial^2 t(E[t],E[s])}{\partial s^2} \cdot \frac{Var[s]}{2}$$

$$= \frac{6E[r]}{17.6\ E[s]^3} \left[E[s]^2 + Var[s] \right]$$

[Internal name is DUR]

ENDØFJØB

This keyword specifies that no more data remains to be run and execution of VESYS II may now be terminated. Note that any data which are input after the last ENDØFRUN will never be run because that directive is needed to cause processing to begin.

ENDØFRUN

This keyword specified that all data needed for the current run have been read. The program begins execution using the data which have been supplied. Any variable which was not specifically set will retain the value it had in the preceeding run. If this is the first run of a job, an undefined variable will take on its default value, if any.

A job consists of one oe more runs followed by an ENDØFJØB directive. It is important to remember that the ENDØFJØB must be immediately preceeded by an ENDØFRUN (see ENDØFJØB commentary).

GNØRMAL

Array of coefficients in the Dirichlet series which approximates the vertical deformation with respect to time for a static load. The approximating function can be written.

$$d_t = \sum_{i=1}^{n} G_i e^{-\delta_i t}$$

where t is the duration of loading,

d_t is the deformation at time t,

G_i is the i^{th} element of GNØRMAL,

δ_i is the i^{th} element of the DELTAS, and

n is NDELTAS, the number of elements in the DELTAS, GNØRMAL, and GRADIAL arrays.

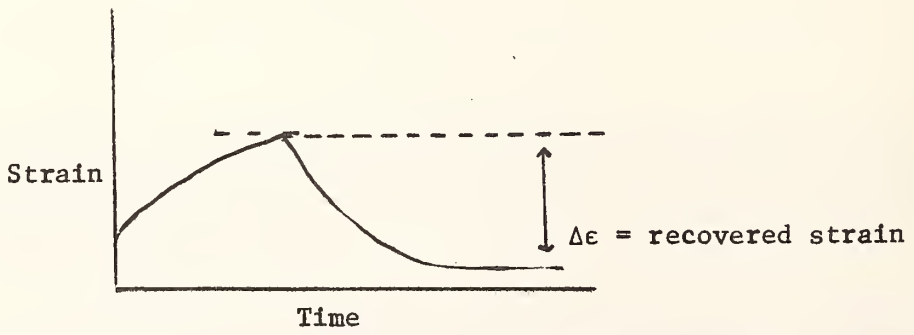
NDELTAS must be input before GNORMAL, the maximum is 20.

This array is passed from the static load subroutine to the repeated load subroutine in a Type 1 run. It need not be input by the user (see TYPE for details).
[Internal array is GNORM(I)]

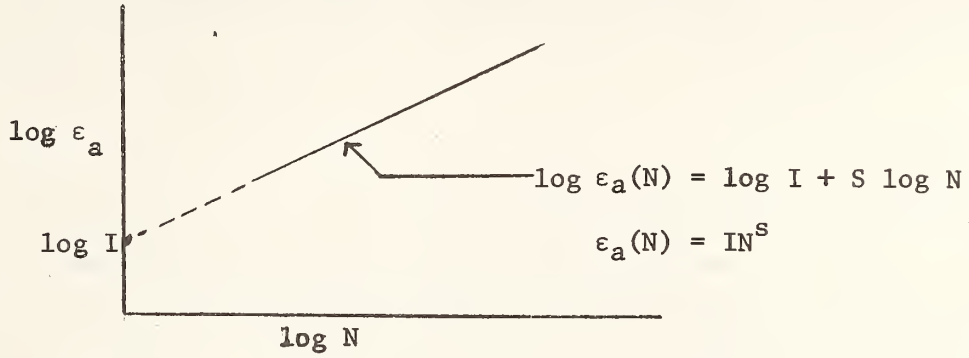
GNU

Array of permanent deformation materials property parameters for the three layers (see also ALPHA). The n^{th} element of this array is the value obtained for layer n.

These parameters are determined from repeated load tests on samples of the materials in the layers. The elastic strain, $\Delta\varepsilon$, for each material is taken after 50 to 200 load applications. The following graph represents a single cycle taken from a repeated load test:



The graph of log accumulated permanent strain versus log number of repetitions is approximated by a straight line:



where

$\varepsilon_a(N)$ is accumulated permanent strain at cycle N,

N is number of load applications,

S is the slope of the line, and

I is the intercept (actual strain, not log value).

The permanent strain due to cycle N, $\varepsilon_p(N)$, is:

$$\varepsilon_p(N) = \varepsilon_a(N) - \varepsilon_a(N-1) \simeq \frac{d\ \varepsilon_a(N)}{dN} = ISN^{S-1}$$

$\varepsilon_p(N)$ is normalized by dividing by the resilient or elastic strain, $\Delta\varepsilon_p$ (measured at about 100 to 200 cycles).

$$\frac{\varepsilon_p(N)}{\Delta\varepsilon} = \frac{ISN^{S-1}}{\Delta\varepsilon} = \mu N^{-\alpha}$$

where

$$\mu = \frac{IS}{\Delta\varepsilon}, \text{ and } \alpha = 1-S$$

μ as defined above is the element of GNU which corresponds to the layer whose repeated load test yielded the values of I, S, and $\Delta\varepsilon$.

The values of GNU for the three layers will be used in the static load subprogram to determine GNUNØRM. This variable is passed to the repeated load subprogram, where it is used to predict rutting and slope variance (see commentary on GNUNØRM).

The values input in the GNU and ALPHA arrays are very critical for the program's response.
[Internal name is GGU(I)]

GNUNØRM

The coefficient, μ, in the system's permanent vertical deformation expression which is used to predict rutting:

$$d_N = d_1 \mu N^a$$

where

N is the number of load applications,

d_N is the vertical deformation after N applications,

μ is GNUNØRM (not the same as GNU), and

α is ALFØRM (not the same as ALPHA).

This is one of the parameters passed from the static load subroutine to the repeated load subroutine in the Type 1 run. It need not be input by the user of the program (see TYPE for details). [Internal name is GMU(4)]

GNURAD

Formerly a part of the system's permanent radial strain expression, this variable is no longer used in VESYS IIM.

GRADIAL

Array of coefficients in the Dirichlet series which approximates the radial strain at the first interface with respect to time for a static load. The function which approximates this response curve can be written:

$$\varepsilon_t = \sum_{i=1}^{n} G_i e^{-\delta_i t}$$

where

t is the duration of loading,

ε_t is the strain at time t,

G_i is the i^{th} element of GRADIAL,

δ_i is the i^{th} element of DELTAS, and

n is NDELTAS, the number of elements in the DELTAS, GNØRMAL and GRADIAL arrays.

NDELTAS must be input before GRADIAL, the maximum is 20.

This array is among the information passed from the static load subroutine to the repeated load subroutine in a Type 1 run. Hence it generally need not be supplied by a user of the program (see TYPES for details).
[Internal array is GRAD(I)]

INDEX (Default is 1.0)

This specifies a reference number for the current run. After each run the value of INDEX is automatically increased by one. The incrementation of this counter may be overriden at any time by inputting a new value for INDEX. If this variable is never input, the runs within a job will be numbered sequentially from one.
[Internal name is INDEX]

ITYPES (Default is 0.0)

This value specifies the kind(s) of information which are to be calculated in a Type 2 analysis. An option is selected by putting a non-zero digit in the appropriate decimal place:

```
                1 .0 = vertical stress
              1 X .0 = shear stress
            1 X X .0 = radial stress
          1 X X X .0 = vertical deformation
        1 X X X X .0 = radial deformation
      1 X X X X X .0 = circumferential strain
    1 X X X X X X .0 = radial strain
```

Any combination of these analyses may be specified. For example, a value of 1001000.0 for ITYPES will cause only radial strain and vertical deformation to be calculated at the points of interest.

In a Type 1 analysis the value of ITYPES is irrelevant, except that it must be non-zero or difficulties arise. For a Type 1 run (both static and repeated load analysis) only the vertical deformation at the point of the load and the radial strain directly beneath the load at the first interface will be calculated. These are the response measures which are needed in the repeated load subprogram.
[Internal name is ITYPES]

K1K2CØRL (Default is 0.0)

Correlation coefficient for STRNCØEF and STRNEXP (frequently called K_1 and K_2 in the literature). These two parameters can be expected to be negatively correlated, they interact to a fairly great extent.
[Internal name is CK12]

LABDATA

The array of data which are to be curve-fitted to a Dirichlet series in a Type 4 run. The array TCURVE represents the time at which the corresponding points in LABDATA are taken.

There must be exactly NTCURVE elements in this array (maximum of 50), and the number of points must be input before the LABDATA array. Fewer than three points should not be curve-fitted because difficulties arise (See TRANDOM commentary).
[Internal array is DATA (I)]

LAMBDA

The array of average truck traffic (axles per day) in the repeated load subprogram. Each element of this array represents the traffic rate at the corresponding point in the TRANDØM array.

There must be NTRANDØM points in this array (a maximum of 25). NTRANDØM must be read in before LAMBDA.

Only the heavier traffic should be counted in the traffic rate, light vehicles should be ignored.
[Internal name is ADT(I)]

LAYER1

LAYER2

LAYER3

Master creep compliance curves for the three layers. Each point in these curves is the value of creep compliance (strain/stress) for the load duration given by the corresponding point in the TSTATIC array. There must be exactly NTSTATIC elements is these arrays, a maximum of 25; and the variable NTSTATIC must be input before any of these arrays.

It is important to note that these are <u>master</u> creep curves, not just a set of laboratory data taken at a particular temperature. It is necessary to perform creep tests at high and at low temperatures and then shift the results to the reference temperature (RETEMP) using the appropriate time-temperature shift factor (see commentary on BETA). This process will yield estimates for creep compliance at very short (.01 to .1 sec) and very long (100,000 to 1,000,000 sec.) load duration.

For purposes of curve-fitting it is advisable to limit these arrays to one ot two points for each order of magnitude of time represented.

There are some difficulties in the curve-fitting routines which prevent inputting less than three points in these arrays (see commentary on TSTATIC).
[Internally these curves are stored in DLAYER(I,1), DLAYER(I,2) and DLAYER(I,3)]

LØADING (Default is 1.0)

In a type 2 analysis this represents the intensity of applied loading in psi.

In a type 1 analysis it is necessary to remember that the value input here will multiply the repeated load variable AMPLITUD. AMPLITUD is treated as a random variable with mean and variance adjusted to reflect the expected distribution of wheel loads. This is simplified by setting LØADING to one, causing the system response curves to be calculated in terms of a unit load intensity, so that the values of RADIUS, VCAMP and AMPLITUD can be set to model the repeated load conditions directly.
[Internal name is QQ]

NDELTAS (Default is 1.0)

The number of points in the DELTAS array and any array for which these DELTAS apply (e.g. CREEP1, CREEP2, CREEP3, GRADIAL, GNØRMAL). DELTAS must be input whenever curve-fitted data are being read in. This is not usually necessary, and the user is cautioned not to attempt to input DELTAS unless their effects on the rest of the system are understood (see relevant commentary). The maximum value permitted for NDELTAS is 20.
[Internal name is NDEL]

NØPART

This keyword specifies that the partial derivitives calculated as intermediate steps in the program are not to be printed out. These values are not ordinarily of interest to a user of the program. This command must be repeated for every run in which this printout is <u>not</u> desired.
[Internal name is NØPART]

NRPØINTS (Default is 1.0)

The number of points in the RPØINTS array, must be input before the array. A maximum of 50 points are permitted.
[Internal name is NUMRS]

NTCURVE

The number of points in the TCURVE and LABDATA arrays. A maximum of 50 are permitted. This variable must be input before the arrays in question. These inputs are only used in a Type 4 analysis.
[Internal name is NTCURV]

NTEMPS

The number of points in the TEMPS array. A maximum of 24 are permitted. This value must be input before the array.
[Internal name is NTEMP]

NTRANDØM (Default is 1.0)

The number of points in the TRANDØM and LAMBDA arrays. A maximum of 25 are permitted. This value must be input before either of these arrays.
[Internal name is NTIME]

NTSTATIC (Defalut is 1.0)

The number of points in the TSTATIC, LAYER1, LAYER2 and LAYER3 arrays. A maximum of 25 points are permitted. If fewer than 3 are input difficulties arise (see commentary on TSTATIC).

This value must be input before any of the arrays whose size it determines.
[Internal name is NT]

PSIFAIL (Default is .1)

The level of unacceptable serviceability. (The length of time required for the expected value of serviceability to reach this point is taken to be the expected lifetime of the pavement (unless this value is not reached within the period of the analysis - see commentary on TRANDØM).

The value which is output for "RELIABILITY" at a given time in the repeated load analysis is the probability that the serviceability of the pavement is greater than PSIFAIL. Since serviceability is assumed to have a Gaussian distribution the reliability will be .5 at the time "failure" occurs.
[Internal name is PSIMIN]

PUNCH

If this keyword is specified, the curve-fitted stress, strain, and deformation data will be punched on cards. There does not appear to be any way to cancel the effects of a PUNCH directive which appeared in a preceeding run of the same job.
[Internal name is PUNCH]

QUALITY0 (Default is 1.0)

The mean serviceability index at time zero. Serviceability is assumed to have a Gaussian distribution. The standard deviation for this distribution at time zero is given by the input variable STDEV0.

Note that this keyword must be spelled with a numeric zero, not an alphabetic "oh".
[Internal name is EM0]

RADIUS (Default is 6.4)

The radius of the applied loading in inches. Since the load is assumed to be circular, this is best thought of as the radius of a circle whose area is equal to the loaded area being simulated. This is important in the repeated load model because the loads are expected as a function of pressure and area:

$$L = \pi r^2 p$$

where

L is load,

r is RADIUS, and

p is tire pressure.

The value of L is actually multiplied, in effect, by the value of LØADING. In a Type 1 analysis, a value of one is often input for LØADING in order to simplify this relationship (see the commentary on AMPLITUD for more details).

The value of RADIUS can also be used to compute an estimate of DURATIØN and VCDUR (see relevant commentaries).
[Internal name is AAA]

REFTEMP (Default is 70.0)

The reference temperature at which the master creep compliance curves in LAYER1, LAYER2 and LAYER3 are given. This variable is

expressed in degrees Farenheit.
[Internal name is TREF]

RPØINTS

The array of radial distances from the center of the load at which static response information is desired in a Type 2 analysis. This input is ignored in a Type 1 run.

The variable NRPØINTS specifies the number of points in this array. It must be input before RPØINTS. The maximum number is 50.
[Internal array is RR(I)]

STDEVO (Default is 0.0)

The standard deviation of the initial serviceability index, QUALITY0, in the repeated load model.

Note that this keyword must be spelled with a numeric zero, not an alphabetic "oh".
[Internal name is SIGO]

STRNCOEF

The value of the coefficient K_1 (at a temperature of $70^{o}F$) in the Miner's law formulation for damage:

$$D_j = \sum_{i=1}^{n} \frac{n_i}{N_i}$$

$$n_i = \lambda_i \, (t_i - t_i)$$

$$N_i = K_1 \left(\frac{1}{\epsilon_i}\right)^{K_2}$$

where

n_i is the number of load applications in the i^{th} time interval,

N_i is the number of cycles to failure in the i^{th} time interval,

D_j is the damage at time t,

ϵ_i is the strain during the i^{th} time interval,

λ_i is the traffic rate during the i^{th} time interval,

t_i is the i^{th} element of the TRANDØM array, $t_o = 0$,

K_1 is STRNCØEF, and

K_2 is STRNEXP.

The value of K_1 varies with temperature. When it is input as an array, each element represents the value to be used at the corresponding temperature from the TEMPS array. Exactly NTEMPS values must be provided. Alternately, the value of K_1 for 70^oF may be input as a single value. In this case the program will estimate the values to use for the temperatures of interest accoring to the following formula:

$$K_1(T) = K_1(70) \cdot 10^{[.08896 - .0023917(T-70)^2]}$$

where

$K_1(T)$ is the value of K_1 at temperature T.

[Internal names are CSAVE, and $K_1(I)$]

STRNEXP (Default is .25)

The value of the coefficient K_2 (at a temperature of 70^oF) in the Miner's law formulation for damage:

$$D_n = \sum_{i=1}^{n} \frac{\varepsilon_i^{K2}}{K_1} \lambda_i(t_i - t_{i-1})$$

This equation is explained in the commentary on STRNCØEF.

The value of K_2 varies with temperature. When it is input as an array each element represents the value to be used at the corresponding temperature in the TEMPS array. Exactly NTEMPS values must be provided. Alternately, the value of K_2 for 70^oF may be input as a single value. In this case the program will estimate the values to use for the temperatures of interest according to the following formula:

$$K_2(T) = K_2(70) - .01349\ (T-70) + .0004624\ (T-70)^2$$

[Internal names are ASAVE and $K_2(I)$]

TCURVE

The array of times used in a Type 4 (curve-fitting) run. They are the times (in sec.) at which the LABDATA are taken.

NTCURVE specifies the number of elements of this array, it must be input previously. A maximum of 50 points are permitted. If less than 3 points are used difficulties may arise (see TSTATIC commentary). [Internal array is TCURV(I)]

TEMPS

Array of temperatures to be used in the repeated load subprogram. These temperatures are taken to represent a yearly cycle. For a given time in the TRANDØM array the corresponding temperature can be found by extending this yearly cycle a sufficient number of repetitions. The year is broken down into equal segments, each temperature being the average for the corresponding time interval.

The variable NTEMPS specifies the number of points in this array. It must be input before TEMPS. The maximum number allowed is 24.

The units of temperature used are degrees Farenheit.
[Internal name is TEMP(I)]

THICK1

THICK2

The thickness of the top and base layers (respectively) in inches. A three-layered system is assumed in VESYS IIM. The subgrade is assumed to be semi-infinite. To model a two-layer system using this program, the materials properties for a pair of adjacent layers should be identical. This will work properly because there is no slippage between layers in the model. An homogeneous half-space could be modeled by inputting the same materials properties for all three layers.

THICK1 may not be set to zero. Difficulties might arise if THICK2 is zero, it would be safer to avoid this.
[Internal names are H1 and H2]

TITLE

This keyword specifies that the next card contains a heading which is to be printed on the output at the beginning of each run until replaced by a new TITLE. If no TITLE is specified, 80 blanks are used.

This input is not found in the original FHWA version of VESYS II (M). It was implemented in the CDC 6000 series version of the program developed by Austin Research Engineers.
[Internal array is ITITLE(I)]

TØLERNCE (Default is 50.0)

Minimum acceptable reliability of the pavement, expressed as a percent. Reliability is the probability that serviceability has not reached PSIFAIL at or before some given time.
[Internal name is TLR]

TSTATIC

Array of times at which creep compliance data (LAYER1, LAYER2, LAYER3) were taken. These times are in seconds. Static load information is printed for each time in the array. These times should cover as wide a range of values as possible, note that the LAYER arrays are master creep compliance curves. Thus it is necessary to obtain low temperature and high temperature creep tests which can be shifted according to the appropriate time-temperature shift factors to yield estimates for very short and very long load durations.

The variable NTSTATIC specifies the number of elements in this array. It must be input before the TSTATIC and LAYER arrays. A maximum of 25 points are allowed. If fewer than 3 points are specified, difficulties arise in the curve-fitting routines (this could be remedied by fixing subroutines MTXINV and CURVE).
[Internal vector is T(I)]

TRANDØM

Array of times at which a printout of the values computed in the repeated load analysis is desired. The variable NTRANDØM must preceed the input of this array, and there must be exactly NTRANDØM time values given. A maximum of 25 times are allowed.

The last value of the TRANDØM array terminates the analysis period. If the expected value of serviceability does not fall below PSIFAIL in this period of time, then the expected lifetime is reported as being at least the terminal time value.

The units of time given in this array are determined by the value of TUNITS (see relevant commentary).

For each time in the TRANDØM array there must be a corresponding value for average daily traffic in the LAMBDA array.
[Internal name is TIME(I)]

TUNITS (Default is 1.0)

Specifies the units of time which were used in the TRANDØM array. An integer from one to six in input according to the following code:

1.0 = seconds
2.0 = minutes
3.0 = hours
4.0 = days
5.0 = months
6.0 = years

TYPE (Default is 1.0)

This is the basic command variable for the VESYS II M program. It specifies which type of analysis is to be performed, determining the subcomponent(s) which will be executed. There are four command codes, permitting the following options:

1.0 = static load analysis and repeated load analysis
2.0 = static load analysis only
3.0 = repeated load analysis only
4.0 = curve-fitting only

Type 1 Analysis:

This calls for a full run which produces predictions of pavement response and expected lifetime on the basis of system geometry, materials characteristics, and environment. The static load subroutine passes primary response information to the repeated load subroutine. Serviceability predictions are then made, based on the primary response information and the various environmental parameters. The following variables will ordinarily be supplied by the user for a type 1 run:

(TITLE)		Control variables
TYPE	1.0	
ITYPES	1001000.0	
INDEX		
NØPART		
UNLOAD		
THICK1		System geometry
THICK2		
LØADING		
RADIUS		
NTSTATIC		Creep compliance
TSTATIC		
LAYER1		
LAYER2		
LAYER3		
VARCØEF1		
VARCØEF2		
VARCØEF3		
STRNCØEF		Fatigue and damage variables
STRNEXP		
CØEFK1		
CØEFK2		
K1K2CØRL		
GNU		
ALPHA		
CØRLCØEF		
CØRLEXP		

Materials properties

QUALITYO STDEVO PSIFAIL	Serviceability bounds	System environment
NTRANDØM TUNITS TRANDØM LAMBDA AMPLITUD VCAMP DURATIØN VCDUR	Traffic variables	
NTEMPS TEMPS REFTEMP BETA	Temperature variables	

ENDØFRUN signals end of inputs, begins execution of run.

The following information is computed in the static load subroutine and passed to the repeated load subroutine:

GNORMAL
CØEFGNRM
ALFNØRM
GRADIAL
CØEFGRAD
NDELTAS

Any values which are input for these variables will be ignored. The previous values will be lost. Subsequent type 3 runs (repeated load only) may use the values which are generated for these variables as default settings (i.e. if nothing in input for them, the values computed will be retained). The advantage of this is that it permits analysis of the same pavement under several different traffic, temperature and serviceability conditions without recomputing the primary response information. A considerable amount of computer time can be saved in this way, because the static load analysis requires a significant computational effort.

Type 2 Analysis:

This calls for static response calculations only. It produces values for stresses, strains and deflections in a three-layered linear viscoelastic system. The kinds of response which are calculated are selected by ITYPES. The points at which response is given are specified in cylindrical coordinates. The depth is given by ZPØINT and the radial distances from the center of the load are given in RPØINTS. Although

a single run only calculates response at a single depth, subsequent runs can be made in which the value of ZPØINTS is altered and the other inputs are left unchanged. The values calculated in a type 2 run cannot be passed to a subsequent type 3 run.

Type 3 Analysis:

It is recommended that type 3 runs be made only after the primary response variables have been calculated in a type 1 run. For a user to input all the required information requires a good understanding of the program. Static response values can be passed directly from a type 1 run to subsequent type 3 runs of the same job. This is explained above for the type 1 analysis.

Type 4 Analysis:

This runs the curve-fitting routines on a set of data. The coefficients and DELTAS for a Dirichlet series which approximates the input curve are printed out. The approximation used is based on a least-squares fit.
[Internal name is MTYPE]

UNLØAD

This keyword specifies that the vertical displacement response factors for unloading are to be computed. UNLØAD must be input in every run for which it is desired. These factors are necessary for any type 1 run. Synthesis for Rational Design of Flexible Pavements, Vol. III (Ref 3)[1] states that UNLØAD is selected automatically in a type 1 run, but this is not the case.
[Internal name is UNLØAD]

VARCØEF1 (Default is 0.0)

VARCOEF2

VARCOEF3

Coefficients of variation for the creep compliance curves.

VARCØEF1 applies to LAYER1
VARCØEF2 applies to LAYER2
VARCØEF3 applies to LAYER3

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[1]Noavenzadeh, F., Soussou, J.E., Findakly, H. K., Brademeyer, B., "Synthesis for Rational Design of Flexible Pavements", Part III, "Operating Instructions and Program Documentation", February 1974.

These values are of importance for determining the coefficients of variation for the system response curves. These in turn affect the damage indicators which are computed in the repeated load subprogram. The effects are likely to be striking, hence it is recommended that some reasonable value be input here. A zero value is liable to give poor results.
[Internal name is DVAR(I), I = 1, 2, 3]

VCAMP (Default is 400.0)

The variance of the repeated load intensity distribution. Intensity is treated internally as tire pressure (in psi), but it is important to the user as a means for modeling the wheel weight distribution of the traffic to which a road will be subjected. Hence the variance of pressure must be adjusted to reflect the variation in wheel weights (which is large) rather than the variation in tire pressure (which is relatively small).

In Chapter 4 of Reference 26[1] Mr. Kenis outlines a good method for estimating the variance of axle weight using a weighting factor to account for the effects of tandem axles. VCAMP can be computed as follows:

$$VCAMP = \frac{10^6 \cdot Var[W]}{4} \cdot \frac{1}{(\pi r^2)^2}$$

Where

Var[W] is the variance of axle weight in kips, and
r is RADIUS (see relevant commentary).

Only the heavier traffic is considered in determining the variance of axle weight, light vehicles should be ignored.

The value of load intensity will be multiplied by the value of LØADING which was used in computing the static response curves (see AMPLITUD and LØADING). If a value other than 1.0 was used for LØADING then the value calculated above for VCAMP must be divided by the square of LØADING.
[Internal name is VCAMP]

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[1]Kenis, W. J., "Structural Analysis Design Guide for Flexible Pavements", Office of Research, Federal Highway Administration, September 1974 (Unpublished working document).

VCDUR (Default is 0.0)

Variance of duration of repeated loadings (see commentary on DURATIØN).

If DURATIØN is taken to be the length of time required to transverse three times the diameter of the loaded area then the following function can be used to find DURATIØN:

$$t(r,s) = \frac{6r}{17.6s}$$

Where r is the RADIUS of the loaded area in inches, and s is the vehicle speed in mph.

Since r and s are assumed to be linearly independent (see commentary on DURATIØN) we can estimate the variance of this function as follows where the partial derivatives of t are evaluated at the means of r and s:

$$\text{VCDUR} = \left(\frac{\partial t}{\partial r}\right)^2 \text{Var}[r] + \left(\frac{\partial t}{\partial s}\right)^2 \text{Var}[s]$$

where

Var[r] is the variance of radius, and

Var[s] is the variance of speed.

Since $\frac{\partial t}{\partial r} = \frac{6}{17.6s}$ and $\frac{\partial t}{\partial s} = \frac{-6r}{17.6s^2}$ we have:

$$\text{VCDUR} \simeq \frac{36}{309.76}\left[\frac{\text{Var}[r]}{E[s]^2} + \frac{E[r]^2\,\text{Var}[s]}{E[s]^4}\right]$$

[Internal name is VDUR]

ZPØINT (Default is 4.0)

The depth at which stress, strain, or displacement information is desired in a type 2 analysis. This value is ignored in a type 1 analysis.

A number of radial points may be calculated in a single run, but they will all be at the depth given by ZPØINT. If different depths are desired for a type 2 analysis of a given system, successive runs may be made changing the value of ZPØINT only.
[Internal name is ZZ]

APPENDIX F

APPLICATION OF VESYS IIM TO BRAMPTON TEST ROAD

Introduction

This work represents that portion of the overall sensitivity analysis of FHWA's VESYS IIM program concerned with application to actual field situations at the Brampton Test Road.

In this case, the application is to the Brampton Test Road in Southern Ontario. The Test Road was constructed in 1965 and periodic observations have been made of its behavior and performance. Moreover, sufficient original data on materials characterization, plus traffic and other information, exists for the application of VESYS IIM in an "original" design manner - i.e., so that VESYS IIM predictions can be compared with actual observed field performance. A complete description of the Brampton Test Road together with an analysis of its behavior and performance is contained in Reference 27[1].

The purpose of this report is to describe the application of VESYS IIM to four selected sections from the Test Road, and to present some initial conclusions from the analysis.

VESYS IIM PROGRAM

The VESYS IIM program was received from Austin Research Engineers on October 21, 1974, on tape and accompanied by two listings. The first listing was from FHWA, written for their IBM 360/65, while the second one was the CDC version run at the University of Texas.

A set of test data was supplied by ARE in order for us to verify that the program was operating properly.

OPERATION OF THE PROGRAM

In our attempts to get the program operating, we encountered some initial difficulty in eliminating compiling errors, mainly due to system incompatability. However, these were relatively minor.

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[1]Haas, R. C. G., N. I. Kamel, and J. Morris, "Brampton Test Road: An Application of Layer Analysis to Pavement Design", Report No. RR182, Ministry of Transportation and Communications, Ontario, November 1972.

The second step involved the use of the supplied data to run the program. We encountered some major problems here in that the echo print supplied was not complete (i.e., didn't represent the "true" echo print) and not all the variables were known (LAMBDA, COEFK1, COEFK2,...). This resulted in incorrect outputs as well as execution errors and considerable time was spent in trying to trace the problem back through the program.

Discussions and consultations with ARE in late January, and receipt of the VESYS IIM Input Dictionary (ARE, December 1974) enabled us to make the necessary logic changes (i.e., use of the UNLØAD command) and incorporate redefinitions (i.e., GNU). We then ran the data from the Penn State Test Track, as used by Kenis, and obtained the same outputs. The program was now operational and we encountered no further difficulties in using it.

Brampton Test Sections Used In The Application Of VESYS IIM

It was decided at an ARE meeting of February 13 and 14 in Austin that four representative test sections from Brampton would be sufficient for the analysis. Consequently, the following were chosen:

a. Section 3 ("full depth")

b. Section 4 ("deep strength")

c. Section 11 ("conventional")

d. Section 17 ("conventional")

The configuration and location of these sections is given in Reference 27. Thicknesses are listed in the following section of this report.

Values of Variables Used For The Brampton Sections

The values of variables used for the VESYS IIM analysis were actual measured quantities where available and were obtained from Refs (27, 28[1], 29[2], 30[3], 16 and 31[4]. Some information was not available

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[1]Haas, R. C. G., "A Method for Designing Asphalt Pavements to Minimize Low-Temperature Shrinkage Cracking," Research Report 73-1, The Asphalt Institute, January, 1973.

[2]Kingham, I., Unpublished Asphalt Institute Test Data on Brampton Test Road Materials, 1966.

[3]Phang, W. A., "The Effects of Seasonal Strength Variation on the Performance of Selected Base Materials," Report No. 39, Department of Highways, Ontario, April 1970.

[4]Kallas, B. F., "The Brampton, Ontario Experimental Base Project," Unpublished Report, Asphalt Institute, September 1967.

i.e., fatigue data (fatigue is not considered to be a major problem in Ontario). Consequently, "average" values for fatigue (as supplied by ARE) were used.

All Brampton data were initially used but the outputs were very unrealistic. So, runs were made incorporating Penn. State Test Track data (as used by Kenis), variable by variable, until reasonable answers were obtained. This resulted when the Penn, State values were used for LAYER1, BETA, GNU and ALPHA of the surface layer material, and Brampton data for everything else. A complete listing of variables and values used is shown as follows:

TYPE 1.0

TYPES 1001000.0

NØPART

UNLØAD

THICK1 - Section 3 - full depth - 11.5 in.

Section 4 - deep strength - 9.5 in.

Section 11 - conventional - 3.5 in

Section 17 - conventional - 3.5 in

THICK2 - Section 3 - (tried values ranging from 0.001 in. - no difference in output)

Section 4 - 6 in.

Section 11 -

Section 17 - 21.5 in. "equiv." subbase

LOADING 1.0

RADIUS 6.5 in. (Brampton)

ZPØINT (Not used in TYPE 1.0 analysis)

NRPØINTS 1.0

RPØINTS 0.0

NSTATIC 9.0

VARCØEF1 0.25 (from ARE)

VARCØEF2 0.25 (from ARE)

VARØOEF3 0.25 (from ARE)

TSTACTIC .01 .1 1 10 100 1,000 10,000 100,000 1,000,000

LAYER1 .15 .55 .74 .144 .235 .302 x 10^{-5} ....(from Penn. State)

Our derived values for LAYER1, using the "indirect" method in the Appendix of Reference 28 were as follows:

.25 .74 2.63 11.9 67

LAYER2 (Brampton)

Section 3 - 3.17 x 10^{-5}

Section 4 - 5.55 x 10^{-5}

Section 11- 5.55 x 10^{-5}

Section 17 - 5.0 x 10^{-5}

These values were obtained from Reference 27 by determining M_r at the center of each layer for the associated deviator stress conditions.

(Note that Penn State values for LAYER2 were all 2.3 x 10^{-5}).

LAYER3 (Brampton)

Section 3 - 3.17 x 10^{-5}

Section 4 - 2.94 x 10^{-5}

Section 11 - 2.86 x 10^{-5}

Section 17 - 2.86 x 10^{-5}

These values were similarly determined from Reference 27 for the appropriate stress conditions.
(Note that the Penn State values for LAYER3 were in the order of 5 x 10^{-4}).

QUALITYO 4.2 (from ARE)

STADEVO 0.3 (" ")

PSIFAIL 2.5 (" ")

RETEMP 70°F(" ")

NTRANDØM 15

TUNITS 5.0

TRANDØM (Brampton)

1 2 3 4 5 6 11 15 16 27
39 40 55 70 87

Note: These times correspond to those when rut depth measurements were made and/or when there were significant changes in traffic.

NTEMPS 12

TEMPS 21.2 20.9 29.1 42.3 53.5 63.7
68.9 67.4 59.7 48.7 36.7 25.5

Note: These are the average monthly temperatures from Brampton, from Reference 16.

BETA 0.162 (Kenis)

Note: Our value is 0.085, which corresponds to ARE's average value.

GNU

Layer 1	Layer 2	Layer 3
Section 3 - 0.38 (Kenis)	0.027 (Brampton)	0.027 (Brampton)
Section 4 - 0.38 (Kenis)	0.40 (")	0.027 (")
Section 11- 0.38 (Kenis)	0.40 (")	0.027 (")
Section 17 -0.38 (Kenis)	0.40 (")	0.027 (")

Note: Our value of GNU for LAYER1 is 0.7.

LAMBDA

Month 1 - 4 333 axles/day
4 -15 " "
15 -39 " "
39 -87 " "

Note: These values are taken from Reference (16).

AMPLITUD 70 psi (Brampton)

VCAMP 0.0

DURATIØN 0.04 sec. (Used in our Brampton studies - i.e., Reference (16).

VCDUR 3.3 x 10 (from ARE)

ALPHA

	Layer 1	Layer 2	Layer 3
Section 3 -	0.765 (Kenis)	0.8852 (Brampton)	0.8852 (Brampton)
Section 4 -	0.765 (")	0.7762 (")	0.8412 (Brampton)
Section 11-	0.765 (")	0.8116 (")	0.8412 (Brampton)
Section 17-	0.7565(")	0.8116 (")	0.8412 (Brampton)

Note: Our value of ALPHA for LAYER1 is 0.71.

STRNEXP 3.1 (ARE)

STRNCØEF 1.33 x 10 (ARE)

CØRLCEØF 1.0 (ARE)

CØRLEXP 0.058 (ARE)

CØEFK1 0.7 (ARE)

CØEFK2 0.7 (ARE)

K1K2CØRL 0.867 (ARE)

ENDØFRUN

ENDØFJØB

Calculated Responses and Comparisons to Measured Values

Table 20 is a summary listing of input variables used according to source and Table 21 shows calculated outputs from VESYS IIM for the four sections (full depth, deep strength, and the two conventional sections). In Table 20, a "K" indicates Penn State Test Track Data, "B" indicates Brampton Test Road Data and "A" means data derived by ARE.

Rut depths and serviceability were periodically measured at Brampton. Figures 83 to 90 show comparisons between the calculated and measured values for rut depth and PSI for the four sections.

Summary and Conclusions

1. Tables 20, and Figures 83 to 90 illustrate that VESYS IIM gives quite realistic values for the conventional sections at Brampton, if the Penn. State Test Track data is used for Layer 1. However, if our "derived" values for Layer 1 are used (i.e., creep compliance versus time relationship obtained by using an indirect estimating procedure - see Ref 28 the calculated outputs from VESYS IIM are completely unrealistic. In other words, if Penn. State properties are used for the asphalt layer, with everything else from Brampton, VESYS IIM gives values for rut depths and PSI which are quite close to observed values for the conventional sections at Brampton.

2. Since the ability of VESYS IIM to predict what actually happens in the field is apparently very sensitive to creep compliance versus time relationship for Layer 1, it is imperative that this be "tied down" very closely. We approached the determination of this property from the point of view of a designer who did not have actual test results - i.e., it had to be estimated and the only method available is the indirect one documented in Reference 28. It might well be argued that our values by this method are unrealistic. Certainly, it seems clear that actual direct testing is needed and that designers should be cautioned not to use stiffness versus time or creep compliance versus time relationships from indirect estimation in VESYS IIM.

3. While direct test result data is needed for creep compliance as a function of time, it is not clear how this might best be determined. There is certainly a need for defining an appropriate test procedure and all the associated variables. For example, Figure 91 shows a creep compliance curve for the Brampton Layer 1 material derived from some dynamic modulus testing

Table 20 Summary of input data source for 16 runs for VESYS 11 (M) on several sections of Brampton Test Road

Variable	Run and Section Number															
	1 Penn. State Test Track (Kenis)	2 Section #4	3 Section #4	4 Section #4	5 Section #4	6 Section #11	7 Section #11	8 Section #11	9 Section #11	10 Section #4	11 Section #4	12 Section #4	13 Section #3	14 Section #4	15 Section #11	16 Section #17
THICK1		9.5	9.5	9.5	9.5	3.5	3.5	3.5	3.5	9.5	9.5	9.5	11.5	9.5	3.5	3.5
THICK2		6.0	6.0	6.0	6.0	24.0	24.0	24.0	24.0	6.0	6.0	6.0	0.008 or 10.0	6.0	24.0	21.5
VARCOEF1		A	A	K	K	A	A	A	A	K	A	A	A	A	A	A
VARCOEF2		A	A	K	K	A	A	A	A	K	A	A	A	A	A	A
VARCOEF3		A	A	A	A	A	A	A	A	K	A	A	A	A	A	A
LAYER1		B	B	B	B	B	K	K	K	K	K	A	K	K	K	K
LAYER2		B	B	B	K	B	K	K	B	K	K	B	B	B	B	B
LAYER3		B	B	B	B	K	K	K	B	K	K	B	B	B	B	B
BETA		B	B	K	B	B	B	B	B	K	B	A	K	K	K	K
GNU(1)		B	K	K	K	B	B	B	B	K	B	B	K	K	K	K
GNU(2)		B	K	K	K	B	B	B	B	K	K	B	B	B	B	B
GNU(3)		B	K	K	K	B	B	B	B	K	K	B	B	B	B	B
LAMBDA		B	B	B	B	B	10 APD	50,000 APD	B	B	B	B	B	B	B	B
ALPHA(1)		B	B	B	B	B	B	B	B	B	B	B	K	K	K	K
ALPHA(2)		B	B	B	B	B	B	B	B	B	B	B	B	B	B	B
ALPHA(3)		B	B	B	B	B	B	B	B	K	B	B		B	B	B
STRNEXP		A	A	A	K	A	A	A	A	K	K	K	A	A	A	A
STRNCOEF		A	A	A	K	A	A	A	A	K	K	K	A	A	A	A
CORLCOEF		A	A	A	K	A	A	A	A	K	K	K	A	A	A	A
CORLEXP		A	A	A	K	A	A	A	A	K	K	K	A	A	A	A
COEFK1		A	A	A	K	A	A	A	A	K	K	K	A	A	A	A
COEFK2		A	A	A	K	A	A	A	A	K	K	K	A	A	A	A
K1K2CORL		A	A	A	K	A	A	A	A	K	K	K	A	A	A	A

Table 21 Calculated output response for Brampton Test Road comparisons.

Outputs @ 1 month	Run Number															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Rut Depth	0.22	7.0×10^{4}	2.8×10^{4}	2.0×10^{4}	-363	5.4×10^{3}	0.07	0.45	0.23	0.17	0.16	0.06	0.05	0.06	0.17	0.17
Slope Variance	0.16	5.6×10^{11}	8.6×10^{10}	5.6×10^{9}	1.4×10^{7}	5.8×10^{9}	0.07	3.2	0.72	0.10	0.09	0.04	0.05	0.02	0.40	0.36
Damage Index	0.03	2.0×10^{8}	1.9×10^{8}	2.8×10^{7}	166	2.3×10^{13}	1.5	7.6×10^{3}	280	0.05	0.05	2.7×10^{-5}	0.10	0.02	280	275
Area Cracked	0	940	940	900	931	925	732	967	962	0	0.0	0	0	0	963	963
Serviceability	4.8	-8.0×10^{9}	-1.2×10^{9}	-5.6×10^{8}	-2.5×10^{5}	-5.1×10^{7}	4.7	3.2	4.2	4.9	4.9	5.0	4.2	4.2	3.6	3.6
Outputs @ Expected	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Life (if not 87)	15 mo.	7 mo.	7 mo.	22.9 Mo	11.9 MO	9.9 Mo		11.4 Mo	70 Mo							
Rut Depth	0.59	9.3×10^{4}	4.4×10^{4}	4.6×10^{4}	-382	9.9×10^{3}		0.47	0.63							
Slope Variance	1.12	9.7×10^{11}	2.2×10^{11}	3.0×10^{10}	1.6×10^{7}	1.9×10^{10}		3.5	5.4							
Damage Index	6.9	4.2×10^{8}	4.2×10^{8}	1.3×10^{8}	200	5.0×10^{13}		9.2×10^{3}	4657							
Area Cracked	1000	988	988	997	948	980		977	1000							
Serviceability	3.6	-1.4×10^{10}	-3.2×10^{9}	-3.0×10^{9}	-2.8×10^{5}	-1.7×10^{8}		3.1	2.6							
Outputs @ Last Trandom (87):	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Rut Depth		1.4×10^{5}	9.7×10^{9}	6.9×10^{4}	-1444	2.0×10^{3}	0.18	1.3	0.67	0.58	0.52	0.20	0.17	0.19	0.45	0.43
Slope Variance		2.3×10^{12}	1.1×10^{12}	7.0×10^{10}	2.2×10^{8}	8.1×10^{10}	0.5	2.5	6.1	1.17	0.96	0.46	0.51	0.20	2.7	2.4
Damage Index		4.2×10^{9}	4.2×10^{9}	6.1×10^{8}	384	4.9×10^{14}	30	1.5×10^{5}	6496	1.2	1.2	1.1×10^{-3}	2.7	0.5	5500	6379
Area Cracked		1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	0	1000	6×10^{-5}	1000	1000
Serviceability		-3.3×10^{10}	-1.5×10^{10}	-7.0×10^{10}	-4.0×10^{6}	-7.0×10^{8}	4.3	-.24	2.4	3.6	3.8	4.6	3.5	4.0	2.5	2.6

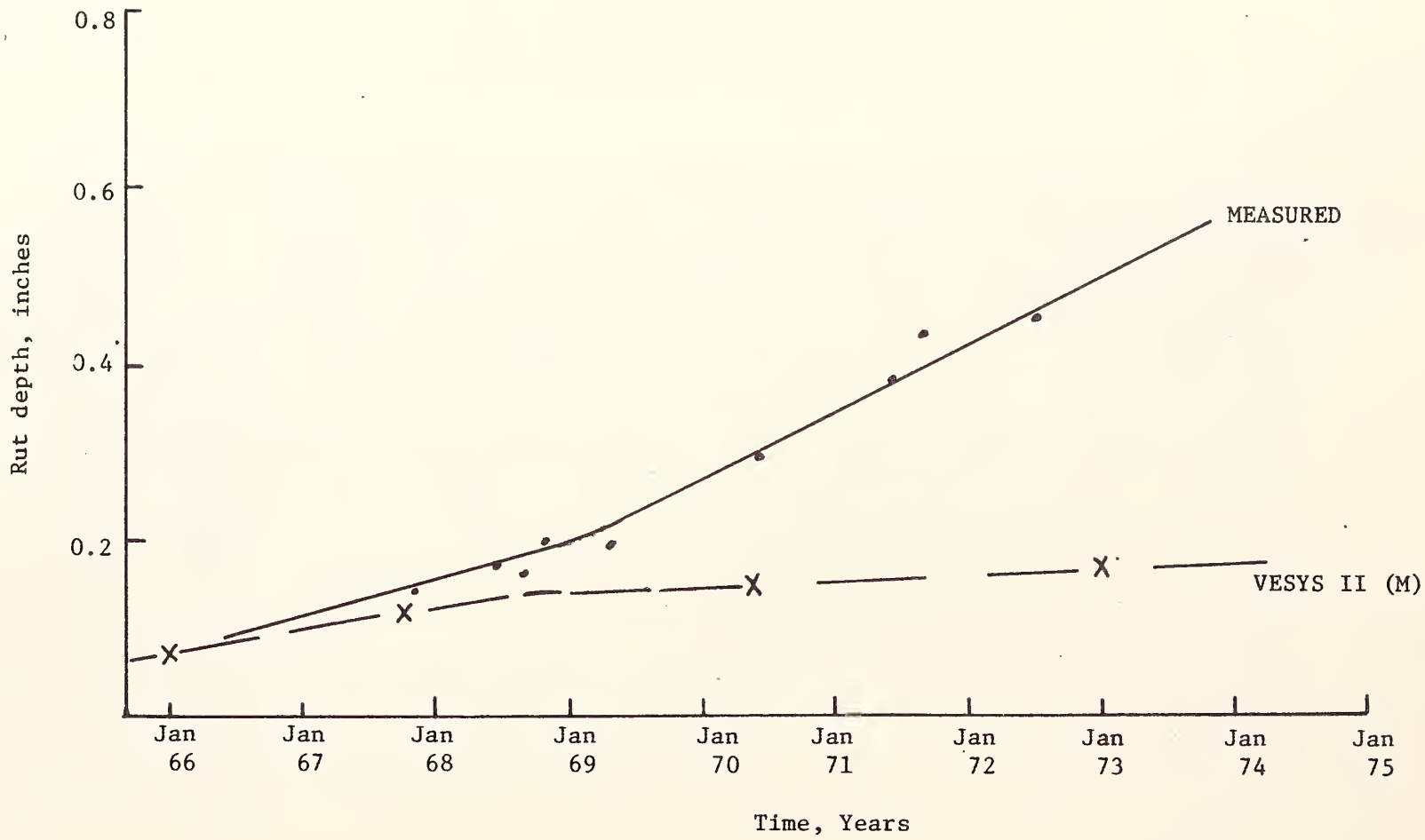

Figure 83. Rut depth versus time, section 3, Run 13.

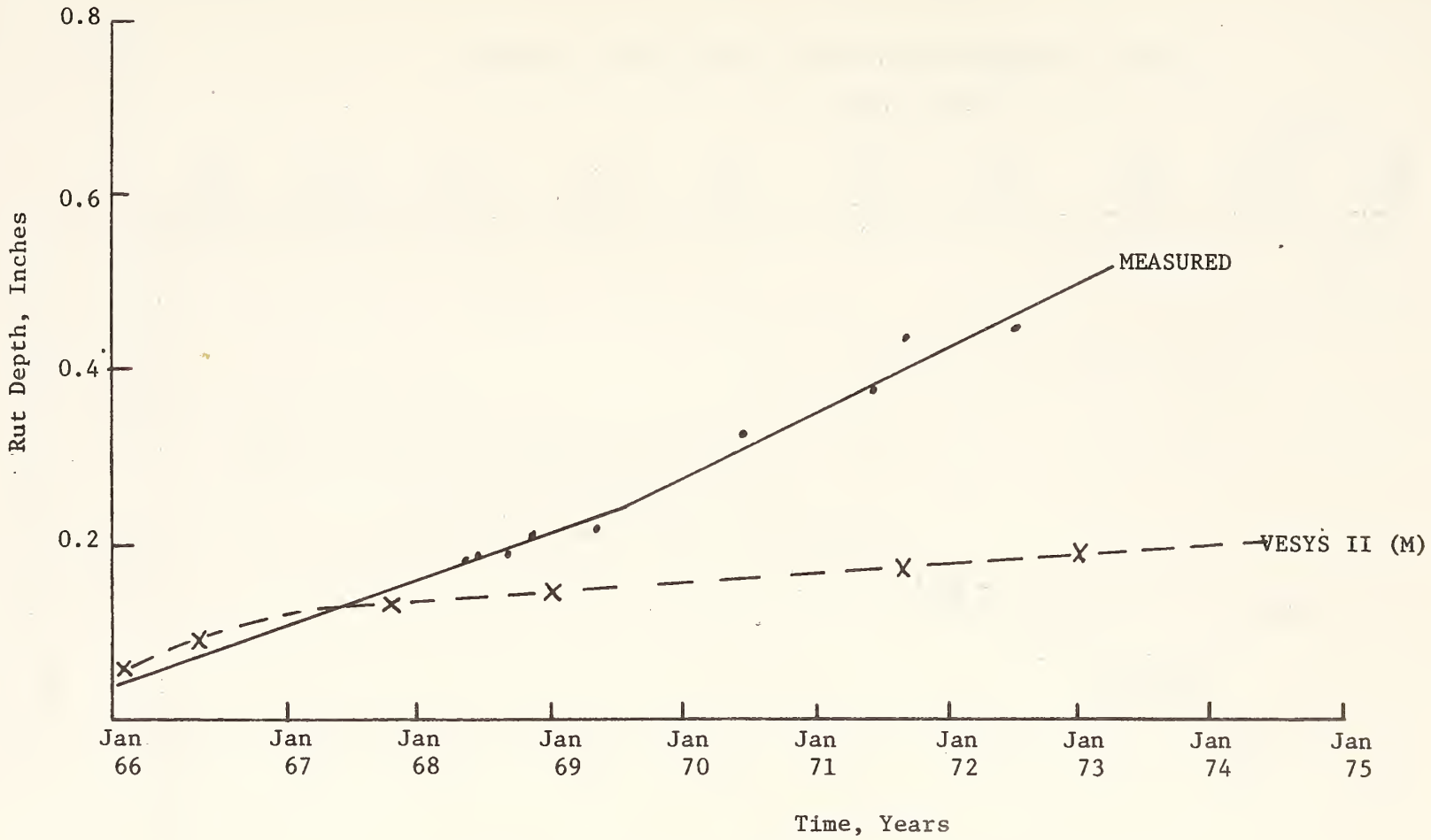

Figure 84. Rut depth versus time, section 4, Run 14.

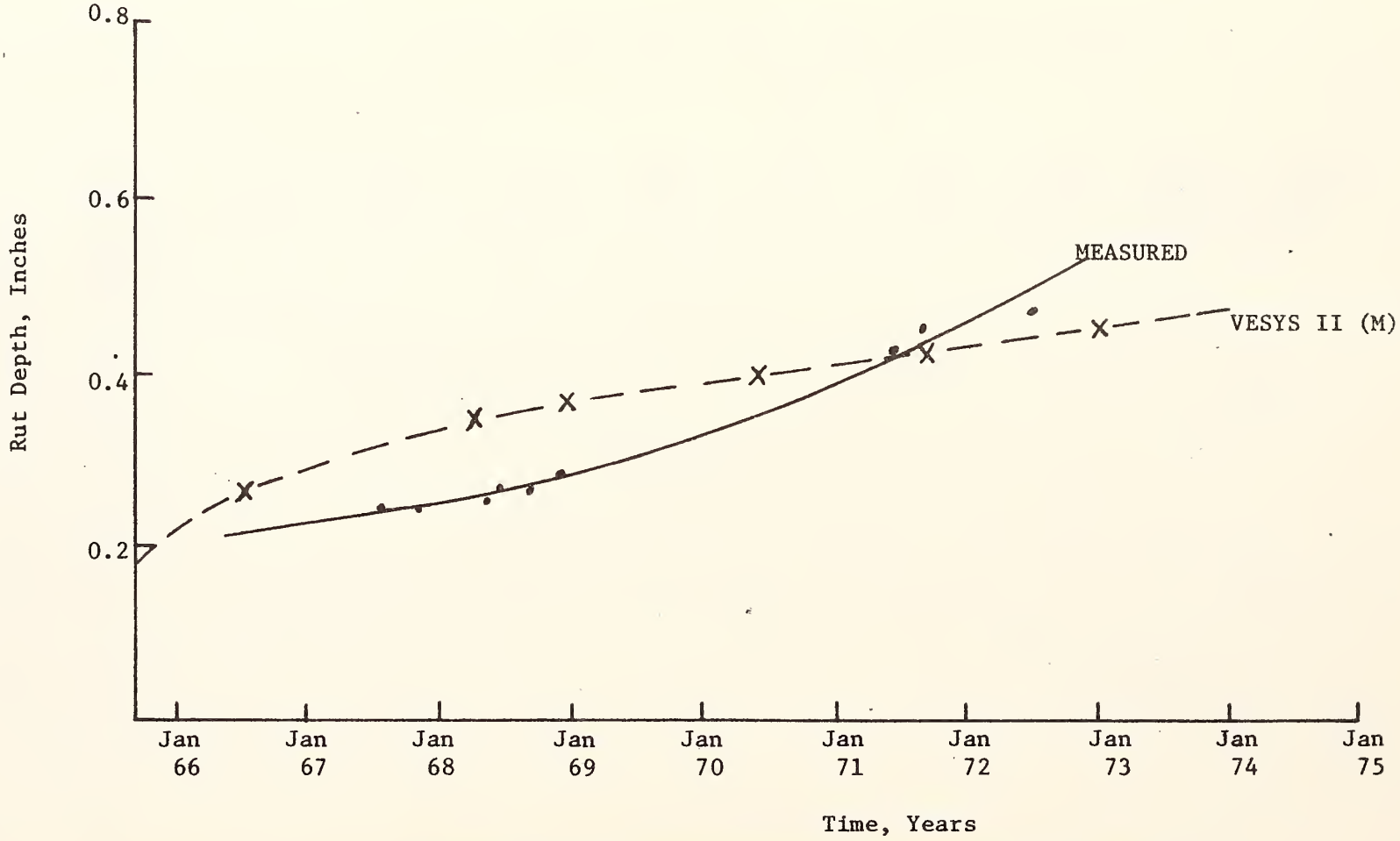

Figure 85.Rut depth versus time, section 11, run 15.

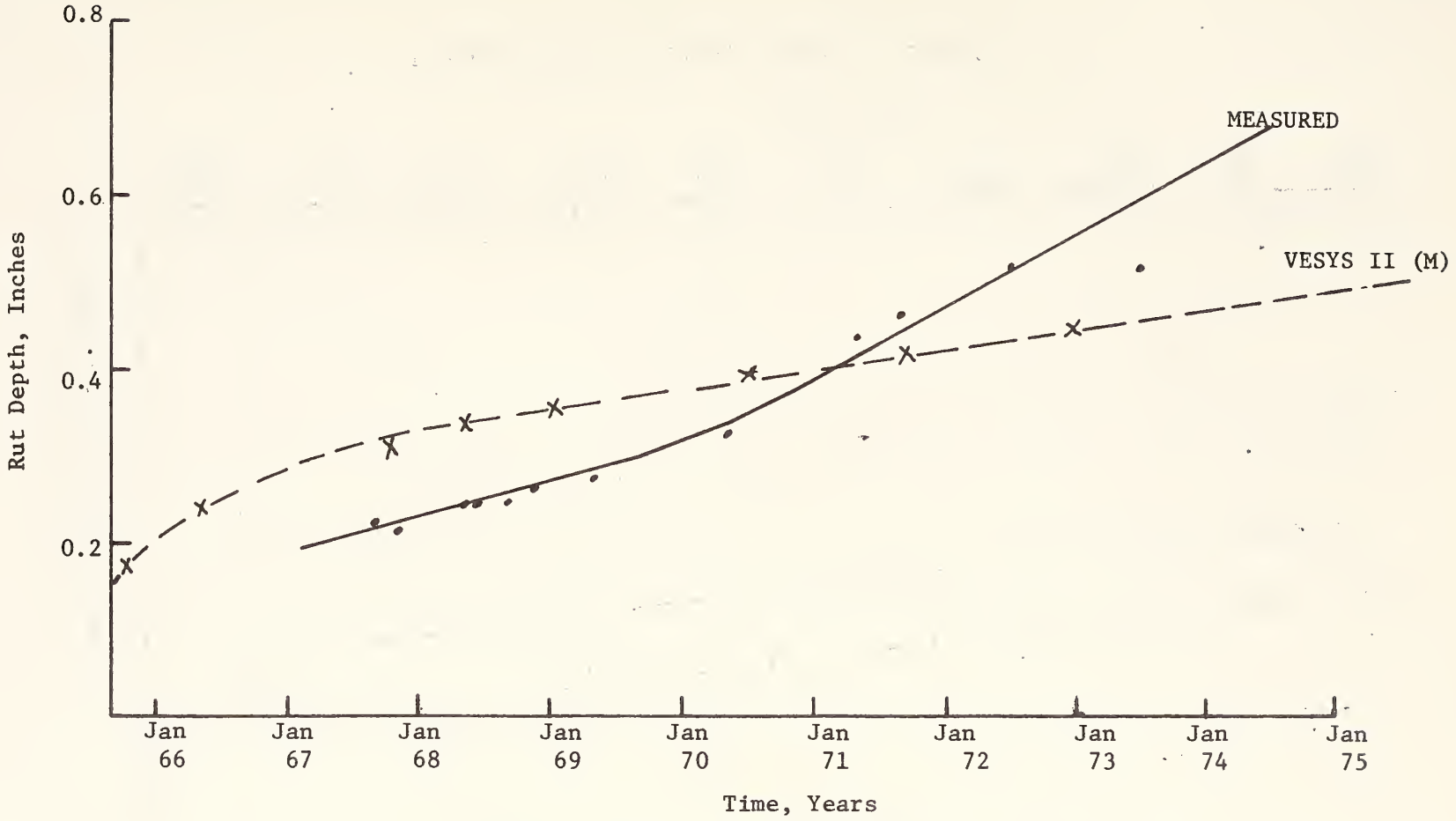

Figure 86. Rut depth versus time, section 17, run 16.

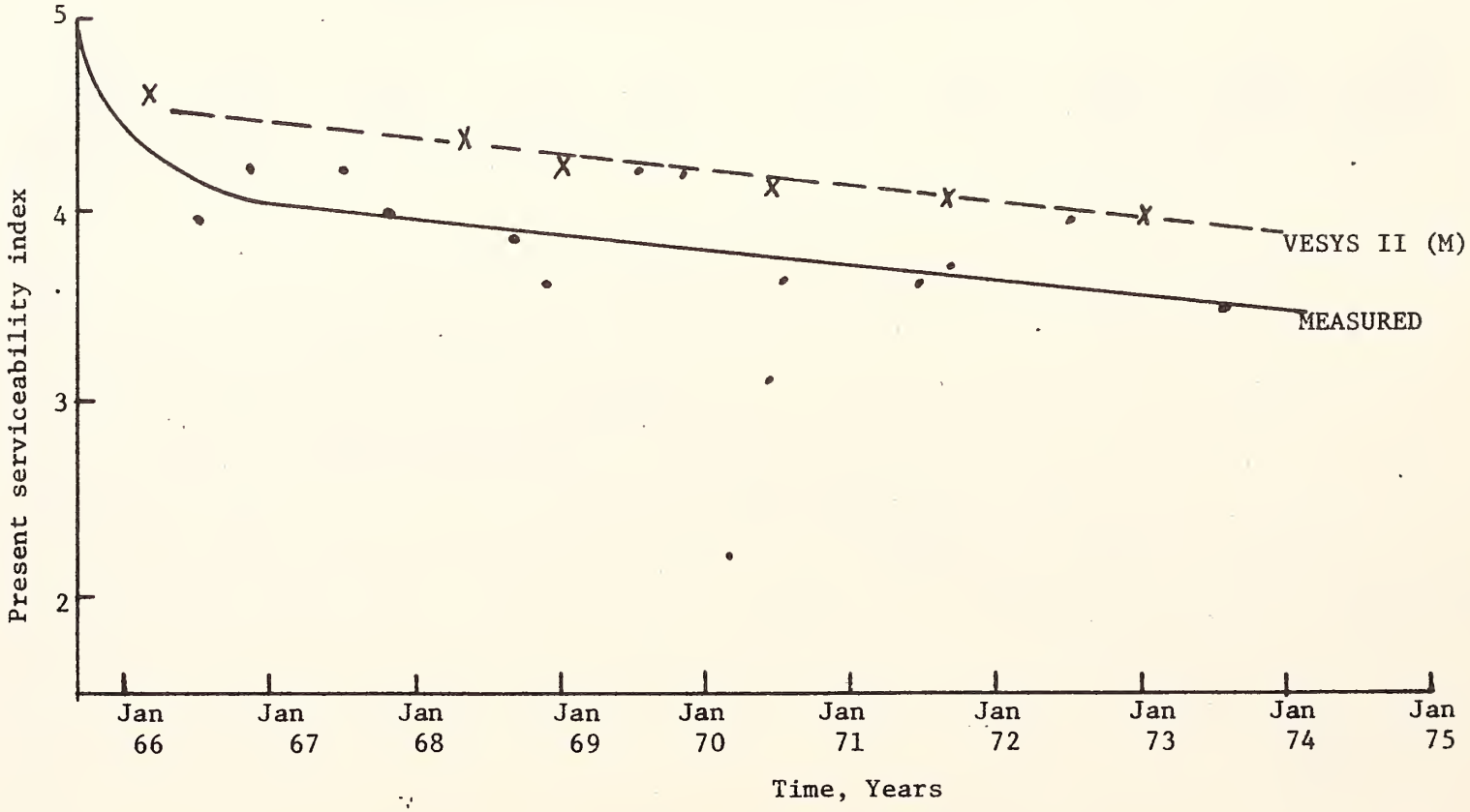

Figure 87. PSI vs time, section 3, run 13.

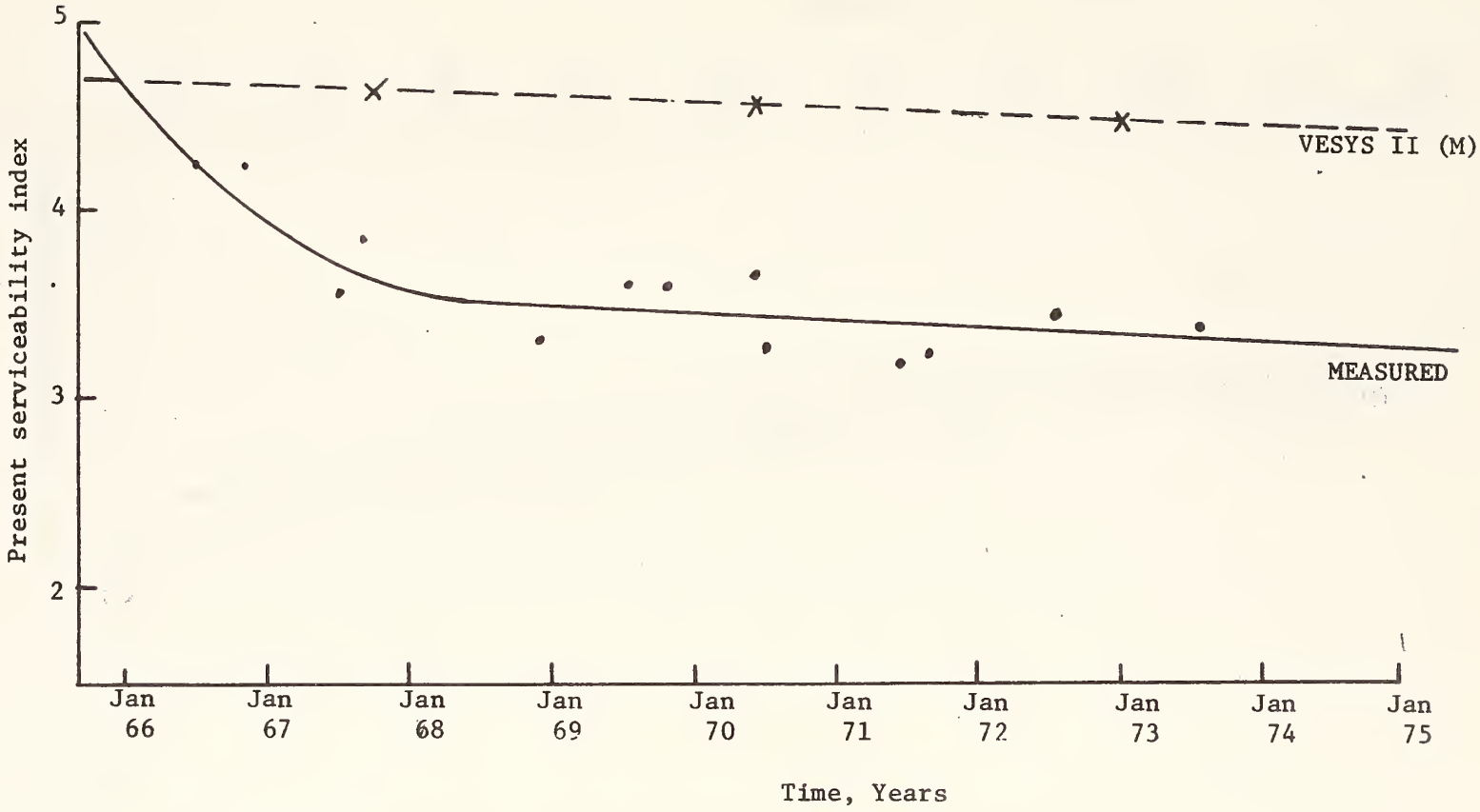

Figure 88. PSI versus time, section 4, run 14.

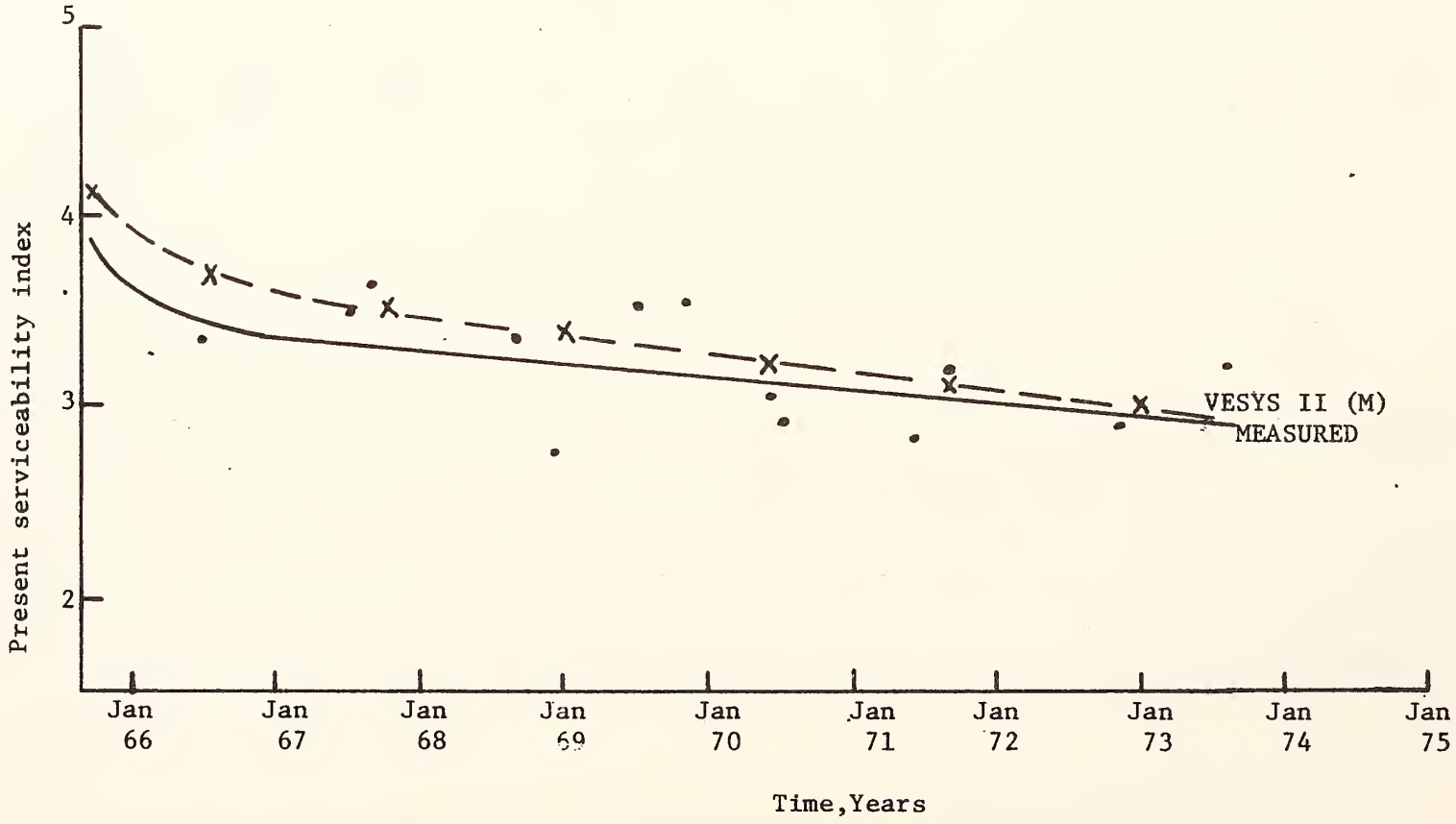

Figure 89. PSI versus time, section 11, run 15.

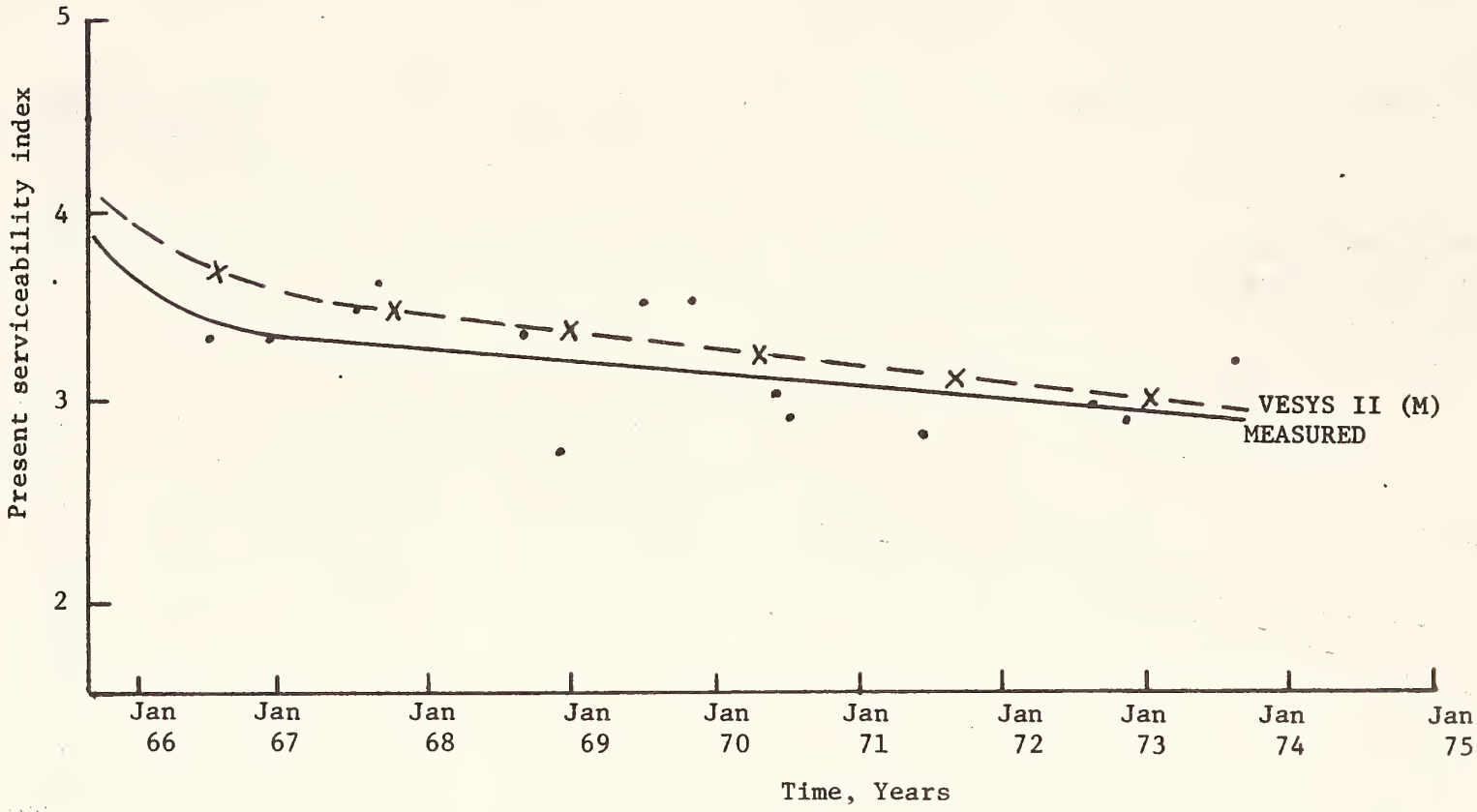

Figure 90. PSI versus time, section 17, run 15.

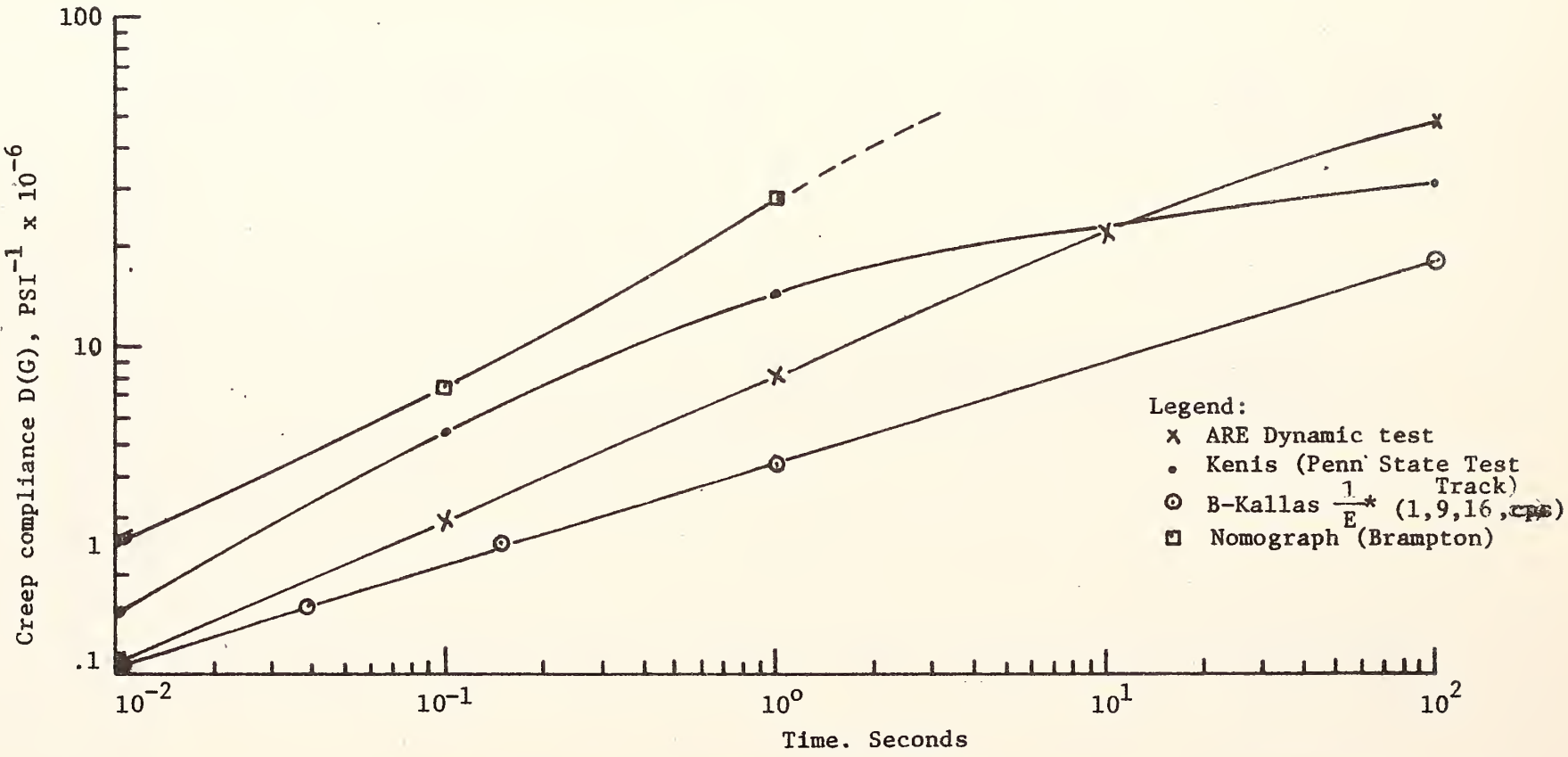

Figure 91. Comparison of Creep compliance curves (asphalt concrete) at a temperature of 70°F.

conducted at the Asphalt Institute in 1967. While Table 20 does not actually show the use of this Asphalt Institute data, it is clear from Run No. 12, that even more unrealistic results would occur because of the low creep compliance (or high stiffness) values. These creep compliance values may well be "too low" in terms of the stress level effects, lack of rest period during testing effects, and so on. Consequently, the "real" Layer 1 creep compliance relationship for Brampton may be somewhere between the extremes noted and perhaps relatively close to the Penn. State relationship. It must be remembered also, that the direct inversion of the Asphalt Institute's dynamic moduli does not give the actual dynamic creep compliance (it would be somewhat greater see Appendix B).

4. For the full-depth and deep strength sections at Brampton, VESYS IIM gives values for rut-depth and PSI which vary substantially from observed values shortly after the construction of the pavement (i.e., after 1 to 3 years), even if Penn. State Layer 1 data is used. This may be partially due to the inability of the Layer 1 data as determined by compressive testing to adequately represent the lower portion of these thick layers which is in tension.

5. The foregoing suggest that:

a. VESYS IIM outputs are very sensitive to Layer 1 properties.

b. These Layer 1 properties have to represent a very sitff mix (an 85/100 pen. asphalt was used at Brampton; the effects of using a softer mix, with say a 150/200 pen. asphalt, may not give realistic results - yet we know that such an asphalt could in fact have been used at Brampton without resulting in undue rutting or serviceability loss).

c. The definition of an appropriate test procedure (as referred to previously in Item 3 above) should take into account the potential change of creep compliance after large numbers of load repetitions. Moreover, it should consider the need for a creep compliance relationship derived from a tension test for the tension zone of the thick layers (see previously referred to Item 4). Which could of course represent a major research program in itself. However, for purposes of the report on this project itself, the dynamic creep compliance testing described in Appendix B looks very promising.

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